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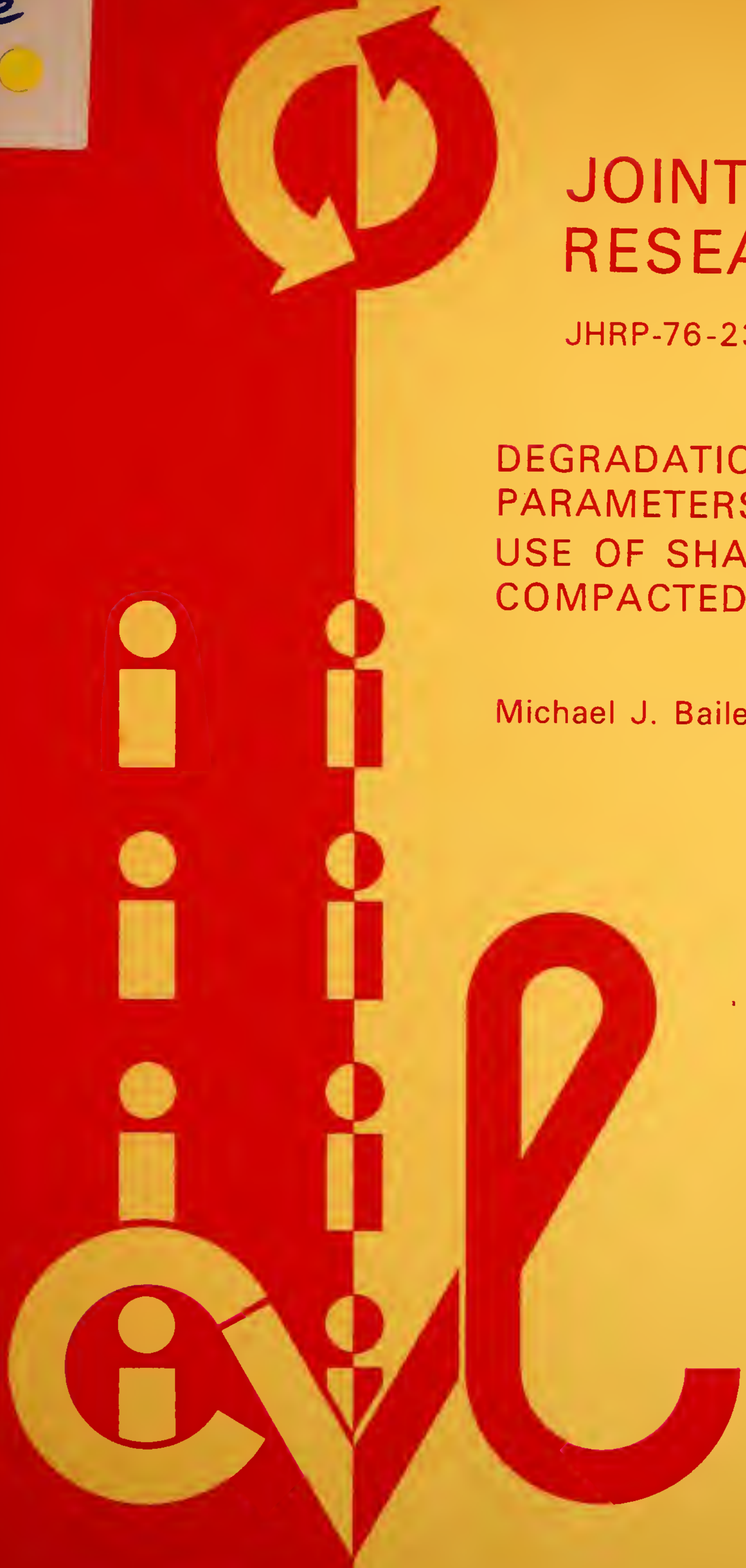

JOINT HIGHWAY RESEARCH PROJECT

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DEGRADATION AND OTHER PARAMETERS RELATED TO THE USE OF SHALE IN COMPACTED EMBANKMENTS

Michael J. Bailey

PURDUE UNIVERSITY
NDIANA STATE HIGHWAY COMMISSION

Interim Report

DEGRADATION AND OTHER PARAMETERS RELATED TO THE USE OF SHALE IN COMPACTED EMBANKMENTS

TO: J. F. McLaughlin, Director
Joint Highway Research Project

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

August 3, 1976

Project: C-36-5L

File: 6-6-12

Attached is an Interim Report on the HPR-1(14) Part II Research Study titled "Design and Construction Guidelines for Shale Embankments". The Report is titled "Degradation and Other Parameters Related to the Use of Shale in Compacted Embankments". It has been authored by Mr. Michael J. Bailey, Graduate Instructor on our staff, under the direction of C. W. Lovell, Jr. and L. E. Wood of our staff and W. J. Sisiliano, Soils Engineer of the ISHC.

The results given in the attached report should be an aid to the ISHC soils engineers and construction personnel in evaluation of shales before their use in highway embankments. Specific tests for such evaluation are suggested and guidelines for reporting pertinent information obtained during shale embankment construction are given so that knowledge on such construction can be furthered.

The Report is submitted as partial fulfillment of the objectives of the Study. Copies of the Report will also be submitted to ISHC and FHWA for their review, comment and similar acceptance.

Respectfully submitted,

Harold L Michael

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Interim Report

DEGRADATION AND OTHER PARAMETERS RELATED TO THE USE OF SHALE IN COMPACTED EMBANKMENTS

by

Michael J. Bailey
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Project No.: C-36-5L

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and the

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The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Purdue University
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16. Abstract

Shales are often viewed with suspicion as embankment construction materials because of their characteristically low strength and poor durability. Typically, shale deposits are nonhomogeneous and problems are encountered in selecting compaction specifications which will insure a uniformly densified fill. Past experience with shales in Indiana suggests that embankment instability is caused at least partly by the presence of large voids, and the collapse of shale pieces into these voids as slaking occurs. It has been hypothesized that the gradation of shale particles before compaction and the degradation that occurs during compaction control the size and distribution of voids within the fill.

Four types of laboratory compaction test have been performed on a model shale to measure the particle degradation and its relation to compaction effort and unit weight. From these results, a static compaction test is recommended as a compaction-degradation index to be used to compare various shales. Additionally, the Scleroscope hardness test and point load strength test have been considered as indices of the engineering properties of shales.

Long term soaking tests on two shales were performed. From these it appears that the effect of moisture is extremely complex. Further research will be required to define this relation and translate it into improvements in the state of the art of shale embankment construction.

17. Key Words
Shale, Argillaceous Rocks, Test Pads, Degradation, Point Load Test, Evaluation Techniques

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This report was typed by Jan Bollinger and Pat Martin. The figures were prepared by Siham Baladi. Special thanks are extended to these ladies and to fellow graduate students David Chapman, Aubrey Abeyesekera and David Todd.

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LIST OF SYMBOLS AND ABBREVIATIONS

AA	arithmetic change in aggregate gradation modulus
AGM	aggregate gradation modulus
C_c	coefficient of concavity
C_u	coefficient of uniformity
D_{10}	effective particle size, i.e. maximum U. S. Standard Sieve size which passes 10% of a sample
D_{60}	maximum U. S. Standard Sieve size which passes 60% of a sample D_{25}, D_{30} and D_{75} defined in same manner as D_{60}
DD	maximum dry compacted unit weight
DM	maximum moist compacted unit weight
G_s	gyratory shear resistance
H	Scleroscope hardness number
IC	index of crushing
N	number of kneading compaction load applications
P	applied compaction load
PA	percent change in aggregate gradation modulus
PLS	point load strength
R	Pearson's correlation coefficient
RN	number of gyratory revolutions
RP	gyratory compaction ram pressure
S_o	coefficient of sorting
t	student's distribution statistic

w	water content
WT	sample weight
$\gamma_{D_{max}}$	maximum dry unit weight resulting from standard Proctor compaction test
10-FACT	ten percent fines aggregate crushing test

HIGHLIGHT SUMMARY

Shales are often viewed with suspicion as embankment construction materials because of their characteristically low strength and poor durability. Typically, shale deposits are nonhomogeneous and problems are encountered in selecting compaction specifications which will insure a uniformly densified fill. Past experience with shales in Indiana suggests that embankment instability is caused at least partly by the presence of large voids, and the collapse of shale pieces into these voids as slaking occurs. It has been hypothesized that the gradation of shale particles before compaction and the degradation that occurs during compaction control the size and distribution of voids within the fill.

Four types of laboratory compaction test have been performed on a model shale to measure the particle degradation and its relation to compaction effort and unit weight. From these results, a static compaction test is recommended as a compaction-degradation index to be used to compare various shales. Additionally, the Scleroscope hardness test and point load strength test have been considered as indices of the engineering properties of shales.

Long term soaking tests on two shales were performed. From these it appears that the effect of moisture is extremely complex. Further research will be required to define this relation and translate it into improvements in the state of the art of shale embankment construction.

The results of this investigation should be an aid in the proper evaluation of shales before their use in highway embankment. An excellent summary concerning current experience in the

construction of embankments and test pads prior to embankment construction is presented. Guidelines are given that will assist in optimizing the knowledge gained from test pads that will be constructed in the future. In fact, each embankment constructed of shale should be viewed as a model for improving the construction of similar embankments in the future. Improved "note taking" and an increased number of control tests can result in every shale fill being a "test pad" for future work. Such information is badly needed when one considers the present state of the art.

A static compaction test is suggested that would be useful in determining a Compactive-Degradation index for evaluating the degradability of shales prior to their use in compacted embankments.

A point load test appears to be an adequate method of measuring the strength of shale pieces. This strength value may prove to be correlatable with such embankment construction parameters as ease of excavation and degradation due to compaction.

The findings of this study should be of primary importance to engineering geologists engaged in the preliminary evaluation of shale material, soils engineers involved in establishing construction guidelines and specifications and to construction personnel that are present during the actual embankment construction.

1. PREAMBLE

1.1 Project Background

The term "shale" is used to label the most common group of rock materials found in the earth's crust. Generally shales have been defined as argillaceous sediments which exhibit the properties of insolubility, slaking disintegration and fissility, but there are a great number of definitions and uses of the term (23)*.

As material to be used in the construction of embankments for highways or dams, shales exhibit an array of engineering properties which can cause considerable difficulty in design and construction planning. Shales are typically nonhomogeneous rocks. A single deposit or formation may have considerable variability in particle size and mineralogy, and/or in degree and type of cementation.

The nonhomogeneous nature of shales, their poor strength and weathering characteristics, and unfortunate past experiences have led many engineers to consider shales as inadequate for embankment construction (23, 49). Shales are considered "problem material" in embankment construction because the current state of the art does not include adequate means of predicting and evaluating the strength and compressibility of nonhomogeneous and/or non-durable fill materials.

The variability in shale properties, and limited durability, require design guidelines based on a quantitative classification of behavior to

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*Numbers in parentheses refer to entries in the List of References, page 191.

enable safe and economical embankment construction. Specifically, tests and evaluation techniques are needed to determine whether a shale deposit can be used as embankment material, and to establish the design criteria imposed by the material's characteristics.

A shale research program was initiated at Purdue University to develop the techniques described above. The principal objective of this research has been to investigate various aspects of shale classification and behavior which can be applied to current embankment construction problems facing the Indiana State Highway Commission and others.

Early shale research at Purdue included a broad spectrum of conventional soil and rock mechanics tests on samples of several Indiana shales (23). More recent work has investigated the current state of the art in shale embankment construction (17), and compared various index tests and classification systems which describe the durability or resistance to slaking of shales (16).

1.2 Outline of Recent Activities

Although primarily a description of laboratory investigations, this report includes a discussion of some recent experiences and problems encountered in construction of compacted shale embankments. Although much work has been done with shale test pads in the field, only a limited amount of this experience has been reported precisely enough to allow engineers to rely on these test results. This problem will be discussed later, in some detail.

Three principal types of laboratory investigation are described in the following chapters. These investigations were conducted to

characterize the engineering behavior of a single shale type and to compare this particular material with other Indiana shales. The standard test material used has been called Shale No. 11, the "Attica model shale" and is described in Appendix A. Tests were performed to measure the particle degradation due to compaction of the model shale, to measure the strength and hardness of various shales, and to monitor the degradation and water absorption characteristics of a shale subjected to long-term soaking.

In any engineering investigation there is the question of how well laboratory tests simulate actual field conditions. Because of this, several compaction tests have been used to evaluate soil and rock materials to be used in embankment construction (46, 47, 92). In the following chapters a point will be made that particle degradation due to compaction is an important characteristic in designing shale embankments. One major purpose for the research described herein was to measure the particle degradation resulting from four types of laboratory compaction tests, at various compaction effort levels. From this, a single test was chosen as a means of classifying or comparing several shales with respect to their degradability. When more information is available from shale embankment test pads and further laboratory investigations, it is hoped that classification of shales' compaction-degradation behavior will provide a means of improving shale embankment design.

The compaction methods investigated in this study were:

a. kneading compaction,

b. gyratory compaction,

c. static compaction, and

d. impact compaction.

These tests are described in detail in Chapter 4.

The second type of investigation reported herein are tests which measured the strength and hardness of shales. The strength of the material may be evaluated in two distinct parts: the strength of the intact rock, and the strength of the compacted mass. Both strength and hardness evaluations must include the effects of various moisture and load conditions.

An analysis of compacted shale strength (stress-strain behavior) is the subject of a companion study. The strength and hardness of intact pieces was investigated because of its potential value in predicting shale construction and post construction behavior. Strength and hardness measurements of shale pieces can be used to evaluate the following:

a. resistance to excavation,

b. resistance to mechanical degradation during compaction, and

c. resistance to mechanical degradation due to load, in the compacted condition.

In each of these cases a range in values for various environmental conditions may be used to correlate the behavior of a given type of shale with material encountered during previous work. If sufficient data are available, strength or hardness measurements can indicate the gross homogeneity or variability of the shale within any given formation. Furthermore, empirical evidence indicates that harder and stronger shales are more resistant to slaking degradation than soft or

weak shales (66). If this evidence is accepted, then some form of strength test may serve for durability classification also.

Current engineering practice does not include any direct relation between the strength or hardness of intact pieces of rock and embankment design parameters. Strength measurements are therefore "classification" tests rather than "design" tests. The different natures of these two designations have been defined by Franklin (28):

> "Classification tests are carried out at the stage of site exploration for the purpose of compiling the basic maps showing the nature of the in situ rock, while design tests are carried out at a later stage to obtain information for use in design calculations."

Obviously such information must be readily available before construction starts. To be of any value, strength or hardness data must be easily obtained and easily interpreted. Tests were performed to measure the Scleroscope hardness and point load strength of a variety of Indiana shales. These tests and their results are described in Chapter 5.

Finally, tests were performed to measure the particle degradation that occurred as a result of long-term soaking of two shale types. These tests were intended to model the slaking deterioration that could follow saturation of a shale embankment.

In reading the test descriptions and results described herein, the major point to be considered is the nonhomogeneous nature of shale. Much more research, and further statistical validation of the research to date, is needed before engineers can confidently and economically construct shale embankments.

2. REVIEW OF CONSTRUCTION EXPERIENCE

2.1 Problems In Constructing Embankments With Shale

In judging the performance of any embankment, strength and compressibility are the factors of most importance. For embankments which may become saturated and for embankments constructed of "problem soils", permeability may also be a major concern. The current state of embankment design indirectly controls these three factors by limiting construction to some region of the moisture-density range for each compacted soil. The selection of a moisture-density range for a given embankment is based on the results of laboratory and/or field tests and is influenced to a great extent by experience and empirical correlations. For "normal" conditions, this design process has been well documented (32, 33, 46, 47, 83, 85, 88, etc.) and needs no further discussion here. For embankments constructed of shale, however, conditions are hardly normal.

As embankment materials, most shales are a cause for concern because they display at least one of the following properties:

1. high swelling capacity
2. fissility
3. non-durability

The characteristic of swelling is primarily related to shales having a high montmorillonitic clay fraction; these shales are typically found in the western U. S. plains and are not of much concern in Indiana.

Fragmented shale particles of silt size are susceptible to extreme swelling through frost action, but may be treated like other frost-susceptible soils.

The characteristic of fissility, and its effect on embankment compaction has been described by Deo (23) and others (24, 93). Briefly stated, the tendency of many shales to break in flat platy fragments can lead to arching or bridging when the material is placed as fill. The result is a nonhomogeneous mix with large irregularly occuring voids. These openings may later collapse as either the material properties or loading conditions change. The subsequent internal embankment movements can be quite serious (93).

The third shale characteristic of concern is non-durability. Shales are liable to degradation through the mechanism of hydrologic dissolution of their diagnetic bonds (slaking), as well as various physical processes (crushing, abrasion, differential thermal expansion, etc.). On a practical level, this means that shale materials may change their properties considerably during the process of embankment construction, as well as over some period of time after construction is completed.

Shale is typically a "soft" rock. Depending on the shale strength, extent of weathering, and the methods of excavation and compaction, a shale embankment can be constructed as a rock fill, a soil fill, or something in between.

Softer shales, which are predominantly broken down to "soil size" during the construction process, can be treated essentially as soil fills (93). It should be noted, however, that the moisture-density

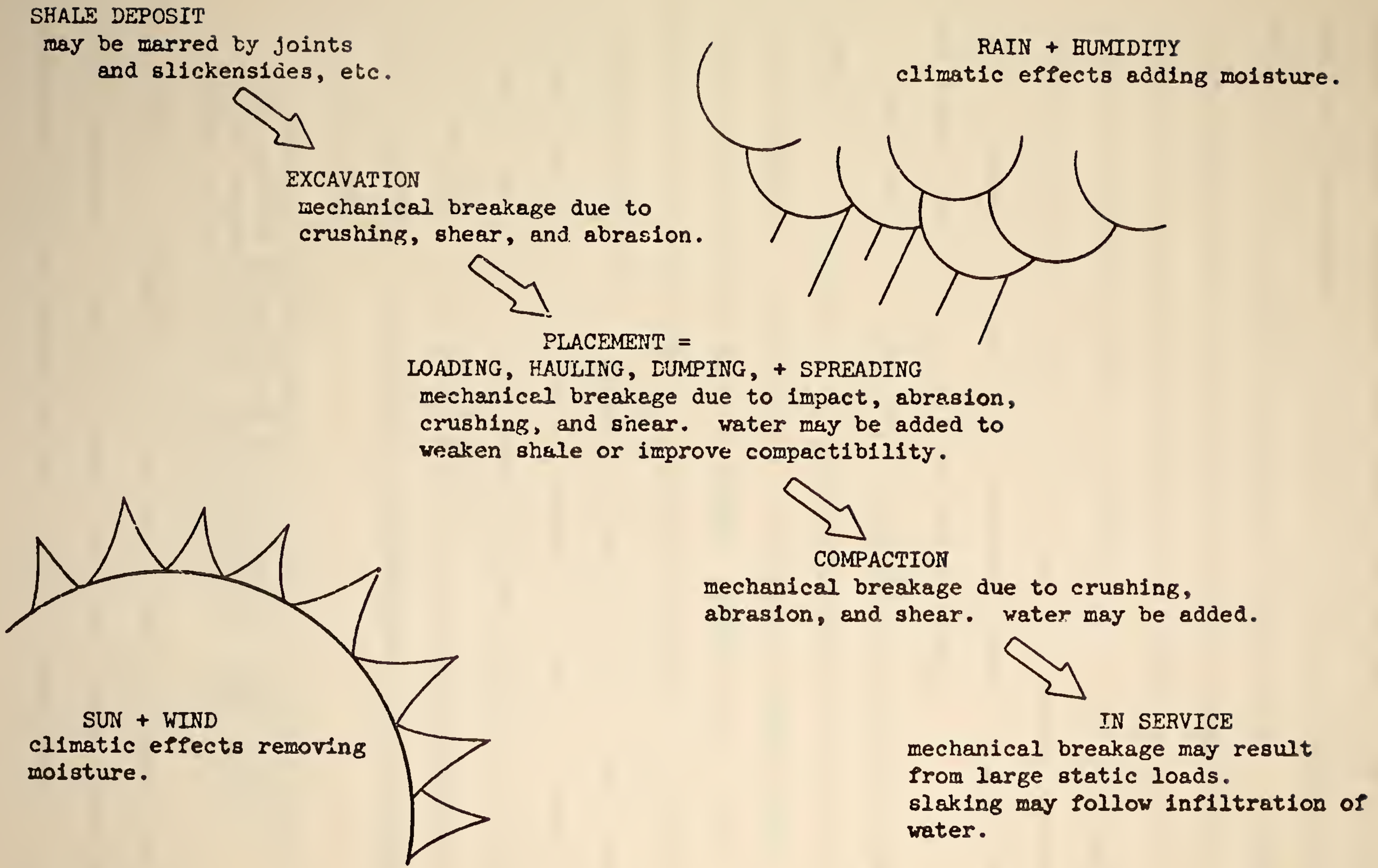
relationship, and attendent parameters of strength, compressibility, and permeability, are considerably influenced by the amount and gradation of any remaining rock size material (90).

Other shales (classified "rock-like" by Deo (23)) have the characteristic of weathering very slowly, and are resistant to mechanical breakdown. These shales, which may be slightly metamorphosed, can be placed as rock fill, with no particular concern about subsequent changes.

Shales with intermediate characteristics appear to be the most difficult to deal with. This general category may include shales which are relatively weak but resistant to weathering, as well as those which are relatively strong but easily slaked.

Figure 1 is a diagrammatic representation of the degrading influences that act during and after embankment construction. Local conditions and construction techniques control the magnitude of these influences. Resistance to degradation depends on a variety of properties. Experience indicates that degradation during compaction depends on: (1) size of the rock fraction; (2) mineralogy and microstructure of the rocky material; (3) gradation before compaction, and (4) construction variables including lift thickness, compaction equipment and effort (90). These same factors obviously affect stability during the other phases of construction also. It is not possible to predict the single effect of any of these factors without prior observation of their mutual interaction (90).

Since even the most durable materials will be degraded somewhat, it is important to consider these influences during the design process.



SHALE DEPOSIT
may be marred by joints
and slickensides, etc.

EXCAVATION
mechanical breakage due to
crushing, shear, and abrasion.

PLACEMENT =
LOADING, HAULING, DUMPING, + SPREADING
mechanical breakage due to impact, abrasion,
crushing, and shear. water may be added to
weaken shale or improve compactibility.

COMPACTION
mechanical breakage due to crushing,
abrasion, and shear. water may be added.

IN SERVICE
mechanical breakage may result
from large static loads.
slaking may follow infiltration of
water.

RAIN + HUMIDITY
climatic effects adding moisture.

SUN + WIND
climatic effects removing
moisture.

Figure 1 Factors Influencing Degradation of Shale Used To Construct Embankments.

In particular, laboratory tests should be performed so as to reflect real field conditions. An important factor in both testing and construction is the natural non-homogeneity of many shales. Irregular sedimentation conditions and subsequent geologic processes have left their effect, sometimes causing extensive variations in material properties and behavior within a single deposit. For this reason tests which depend on a large volume of material, and/or analyses which are amenable to statistical treatment, appear to provide the best means of predicting actual compaction behavior.

What kinds of problems occur during the construction of shale embankments? Chapman and Wood (17) have noted that about half of the agencies engaged in constructing shale fills have trouble mixing water uniformly into the shale to improve compactibility. Some agencies try to handle this by extensive watering on the grade (18, 74), sometimes using disking equipment to improve mixing. Other experience indicates that normal rainfall can improve the moisture content by two or three percent (50), or that relatively dry compaction is acceptable if good drainage is provided to protect the embankment during its service life (74). The problem is well described by this experience from Alaska (1):

> "It does not appear practical to attempt to add moisture to this material in the field for purposes of compaction, as in its fragmented state the claystone will drain freely rather than absorb the added moisture."

This appears to be confirmed by the wide variability noted in moisture contents measured in test fills described by the Indiana State Highway Commission (ISHC) and others (40, 42, 74, 91). Furthermore, at Cachuma

Dam, material was separated and a test pad constructed using the fraction passing a three-inch screen. Water was added at the separation plant under controlled conditions, but field measurements indicated a range in moisture content of 13 to 15 percent "...despite efforts to obtain a homogeneous soil" (33).

Another problem has been the difficulty of adequately degrading the shale during the construction process. This is a factor in obtaining uniform density. The purpose of controlled degradation is two-fold: (1) to eliminate large plate-shaped fragments which tend to bridge over voids, and (2) to provide enough fines to insure voids are filled. If there are sufficient fines, adequately densified, filling the voids between the larger pieces of shale, the collapse of these large pieces following slaking will not cause large internal settlements in the embankment. Thus, in addition to slaking tests (16, 23), tests to monitor the particle gradation during compaction seem called for in shale embankment design.

The U. S. Soil Conservation Service (SCS) (90) has noted that for moderately durable and non-durable rock materials (including but not limited to shales), the percentage of material passing the number four sieve size, after compaction, is a good indicator of the fill's properties. A three part rule of thumb has been developed and is presented in Table 1. Such "rules" are an integral part of soil mechanics practice, but frequently offer as many questions as answers. A case in point is the problem of a non-homogeneous mixture of more and less durable shale. Where only part of the material can be degraded to soil size, the SCS recommendation notes that the engineering properties

Table 1.

Influence of The Plus No. Four Sieve Fraction On The Compacted Soil Mass.
After U. S. Soil Conservation Service (90).

Size Range After Compaction	Engineering Properties	Design Procedures And Treatment	
		Moderately Durable Rock	Nondurable Rock
65% or more passing #4 sieve.	Generally controlled by the compacted characteristics of the minus four fraction.	For relatively important or critical structures, design and control should be based on performance of test fills. For embankments of low hazard or less relative importance, design and control will be based on the compacted density of the mass.	Predictions regarding breakdown and compacted characteristics of these materials will be made on the basis of test fills or documented experience with similar materials. Design and compaction characteristics will be based on mass density in accordance with laboratory tests covering a range of breakdown and densities when test fill information is not available.
35% to 65% passing #4 sieve.	Significantly affected by the size, gradation, amount and character of the plus four fraction, as well as the compacted characteristics of the minus four material.		
35% or less passing #4 sieve.	Related to placement conditions, overall density of the mass, and character of the plus four fraction. Generally not enough fines to adequately fill voids.	Design will be based on engineering properties of the mass from special tests and correlative experience. Placement and compaction will be controlled by method specifications.	

will depend on the character of the oversize rock, the gradation, and the usual compaction characteristics determined in standard laboratory compaction tests. This leaves unanswered the questions of how much degradation will occur during compaction, and whether degradation can or should be increased during construction.

Deo, Wood and Lovell (24) note that shale fills should not be a mixture of relatively durable and non-durable materials. The problem is that of differing rates and magnitudes of degradation, with no way of assessing the properties of the mass. Deo et al. propose "...if it is not possible to separate good and bad shales, then the whole material should be treated like a soil, i.e. be thoroughly broken down" (24). This attitude has been corroborated by other experience (74) with special emphasis on problems which arise when relatively competent sandstones and limestones are interbedded with soft shales. However, this is not a universal problem. A recent survey of embankments constructed with shales reports: "Adequate breakdown of most clay-shales will occur during normal excavation and placement procedures" (18). Other experience indicates that the suitability of shale can be improved by adding "a local granular material" to improve the grading (54).

Where conditions have required it, special breakdown and compaction techniques have been developed to insure uniform density in the completed embankment. For shales which are susceptible to degradation and loss of strength due to saturation and slaking, the absence of "large" voids would seem to increase embankment stability, although that is by no means a certainty. These special techniques are discussed later.

The U.S. Army Engineering Waterways Experiment Station (WES) is currently involved in a 4 year project financed by the Federal Highway Administration that has as its goal the following objectives:

a. Identification of factors responsible for the deterioration of compacted shale.

b. Development of techniques to evaluate the stability of existing compacted shale embankments.

c. Development of remedial treatments for existing distressed compacted shale embankments.

d. Development of design criteria and construction control techniques for compacted shale embankments.

The first three objectives have been completed and presented in two reports (95,96) that are now available. A third report covering objective d should be available within the next year. With the completion of this project, many of the problems associated with the proper use of shales in compacted embankments should be better understood.

2.2 Information From Test Pads

Some mention has been made of the use of shale test pads, i.e. large scale model fills, to determine design and construction guidelines for further construction. Three considerations seem paramount:

1. Why build a test pad? What kind of information can be gained?
2. What variables influence test pad results?
3. What information has already been learned using test pads?

These questions must be considered carefully during the planning and implementation of a new test pad, as well as in evaluating and applying the results of previous work. Thorough documentation of all parts of the project is necessary, too. There is an unfortunate amount of incomplete information in the existing literature.

In a test pad the engineer is able to control and explore the significance of the same variables that exist during normal construction. The test pad in the field enables this control and exploration without the limitations and effects of scale that are present in laboratory tests. There is a tradeoff though, in that laboratory tests can be more easily controlled to examine the influence of just one independent variable at a time. Table 2 is a comparison of the limits and assets of field and laboratory compaction-degradation tests for shales. An important point to note is that laboratory tests cost less and can easily be repeated if ambiguous results are noted. This suggests that a good reason for building a test pad is to gain information relating field performance to laboratory behavior. The object of the test pad is to observe the reaction of the material as it is affected by the

Table 2.

Assets And Limitations of Laboratory And Field Compaction-Degradation Tests On Shales.

Area of Consideration	Laboratory Test	Field Test	Examples
Material Homogeneity	Easy to control homogeneity through selective procedures in preparation of small samples. However, relatively small samples may not reflect actual range in material condition and properties.	Large amount of material involved makes selective control very expensive or impossible. However, results are likely to reflect real conditions.	1. Initial particle gradation 2. Compaction moisture content
Experimental Scope	Very small scale reactions can be measured. Mechanistic behavior can be delineated and quantified.	Size of sample favors observation of bulk properties. Reactions within sample can only be described by extrapolating point measurements.	1. Changes in particle gradation 2. Particle strength
Environment	Easily controlled, but simulation is restricted by physical properties of test.	Little or no environmental control, but physical properties are same as for actual construction.	1. Temperature and humidity 2. Compaction mold effect
Time and Repeatability	Virtually no limit. Rate of testing and number of tests easily varied. Test conditions can be identically repeated.	Rate of tests is same as rate of actual construction. Stand-by time costs for men and equipment may limit number of tests.	1. Slaking 2. Rate of densification
Cost	Low permanent personnel requirement. Tests generally cost little except for initial equipment investment.	High mobilization and use cost for equipment and personnel, for each test pad.	

construction equipment and procedures that are on trial. If this can be correlated to the results of simple laboratory tests, the test pad will then pay for itself by eliminating the need for, or reducing the scope of, test pads on future construction jobs with similar materials. This relation of laboratory tests to field conditions has been noted in recent procedures developed for shale test pads by the ISHC (40, 79).

A second good reason to construct a shale test pad is to evaluate the performance of excavation and compaction equipment. While Chapman and Wood (17) have reported that the method of excavation is generally left up to the contractor, it should be noted that substantial job savings may be realized if the contractor can base his bid on actual field experience with the shale in question.

A case in point occurred during a test fill conducted by the SCS. On this job, the contractor ran seismograph studies from which he concluded the shale could be excavated with a single bulldozer-mounted heavy duty ripper. The test pad revealed that while this method of excavation was possible "...a push cat would be necessary for an efficient large scale operation" (91).

Similar experience resulted from another test pad, this one constructed by the ISHC. While the shale encountered had been properly characterized by the preliminary investigation, the test pad construction revealed the presence of limestone layers interbedded with the shale which greatly hampered excavation. Ultimately, two bulldozers were needed to assist each scraper. The test pad also indicated that attempts to increase degradation of the shale-limestone mixture, using both disking and raking the larger fragments off the lifts, were

of little value for this material (40).

Other test pads have revealed similar problems in achieving the uniform compacted densities called for in the job specifications. Since the shale material does degrade during the compaction process, and because the influence of the "oversize" particles is relatively unpredictable, the engineer is faced with a significant problem in specifying the density to be achieved or the magnitude of effort to be applied, in compacting a shale embankment. For this reason, many shale test pads have served primarily as compaction equipment trials. While an engineer may know that several shale embankments have been successfully built by rolling each lift with three to six passes of a fifty-ton rubber tired roller (74); he may also know that type of equipment isn't available for his job. Field tests may be needed to predict the performance of available equipment.

One shale test pad demonstrated that controlled routing of the loaded dumptrucks produced better results than did a roller that was being tested. Further tests, one year later on the same job, resulted in changing the compaction equipment again. After extensive tests an 8-1/2 ton vibratory roller was selected for use on the remaining 1,750,000 cubic yards of shale fill. The tests showed that "...the roller would generally give densities that would be acceptable even without regular control testing" (50).

The results of a test pad can help the engineer prepare realistic specifications for the larger job that follows. To compact a shale material adequately, the test pad may show whether a particular piece of equipment should be specified, a general class of equipment (say, "a

tamping roller, minimum weight 60,000 lb."), or whether the choice of equipment is relatively unimportant for the project in question. Whether the specifications are to be of the method or end-result type, a test pad can produce a measure of the variability in material response that will occur. Assessing the extent and nature of this variability may lead to changes in the designed safety of the completed embankment.

In a similar way, the test pad experience can tell the engineer how much water, if any, should be added to the shale to aid breakdown and improve compaction (42, 91). A test pad can determine whether existing construction inspection procedures are adequate or whether more control is needed.

Problems involving effective utilization of construction equipment can be minimized if the proper information is available from a trial excavation and test pad. Unlike some borrow materials, shales may include a significant range in strength or density within a single deposit. This can affect the work rate in different parts of the construction schedule. A test pad can expose equipment shortcomings and enable more efficient planning. Large savings can be had if empty trucks don't have to wait on the excavation equipment, or full trucks wait to get on to the lift, etc. There is no reason for an engineering report to conclude with: "No data is available from which to definitely predict the effect of reducing the number of rolling passes" (84). Those unavailable data have potential impact on reducing equipment wear, improving compaction, and saving time and money.

What variables influence test pad results? In constructing a shale test pad, certain observations and controls must be included if the measurements made are to be of any value. The object of the test pad is to provide measurements and experience that are relevant to the actual construction project (18). Experience in Indiana has indicated that the following independent variables must be considered (40):

a. shale type and initial condition,

b. roller rating,

c. number of coverages,

d. placement moisture content, and its variability,

e. lift thickness and number of lifts,

f. foundation material type and condition, and

g. weather.

A good description of the shale type and initial condition is needed because the borrow material may change during later excavation in the same formation. If this occurs, the changing conditions will at least be well documented, and the initial test pad results may indicate the changes necessary in construction procedure. In the short run such information may aid in settling "changed-conditions" claim disputes between owner and contractor. In the long run, such information may eliminate the need for a test pad if the same or similar material is to be used on another project.

Information on initial conditions is also essential if laboratory tests are to be developed to simulate field work. Experience on several projects (18, 40, 90) indicates that the most important information in the material description is in either of two categories:

a. Description of material as it reaches the fill; i.e. the size and gradation before compaction, and the efficiency of the excavating, hauling, and placement equipment in breaking down the shale.

b. Description of undisturbed material; i.e. the thickness and relative hardness of any visible laminations, joint frequency, and degree of weathering.

A good description of the equipment and the way it is used is also important. It is especially important to note any modifications to the equipment, since many contractors try to get the most use from a piece of equipment by "improving" it. These modifications may help, or hinder, or have no effect on construction of a shale embankment, but they should be noted for future reference (33). Notes taken at a test pad should include qualitative comments on the way the equipment performed. Such things as, whether a disker rode over instead of breaking up some of the larger pieces, or, on which pass did the sheepsfoot start to walk out, can help in later evaluation of the work.

Variations in moisture content and related effects have been discussed elsewhere in this report. How and when the water is added during the construction sequence may become a fundamental element in improving the breakdown and compactability of shales. At the present time the evidence on this is insufficient. For this reason special care is needed in recording and using such information from a test fill.

Also discussed elsewhere in this report is the effect of the foundation on degradation and densification. Soft zones in the

foundation may cause non-uniform densification and degradation during compaction. Later collapse of the resulting voids in the lowest layers of the embankment may result in substantial strength loss. Field tests are needed to determine the effect of the foundation on any given shale compaction procedure.

Finally, what information has already been gained using test pads? In considering the results obtained from shale test pads, the non-homogeneous nature of the material as well as the interaction of the effects of moisture, compactive effort and particle gradation must be considered. It is difficult to separate the effects of these variables unless the test pad has been planned so as to minimize overlap. As the number of variables increases, so must the number of tests that are to characterize or reflect the variables' influence. The tests generally consist of unit weight, moisture content, and particle gradation determinations. Statistical analyses enable the segregation of data into primary categories of typical and non-typical response. Evaluation of the typical responses, measured after changing each independent variable, characterizes the shale behavior. This characterization of behavior may be used to design compaction specifications which reflect the real equipment capabilities for the shale in question.

An example of this is shown in Figure 2, which shows the range in compacted density as a function of the initial lift thickness. This information was obtained by the SCS (91) from a shale test pad constructed in Indiana in 1968. For this test, the compaction effort was held constant, but the moisture content and initial particle gradation were variable. The combined influence of these variables is

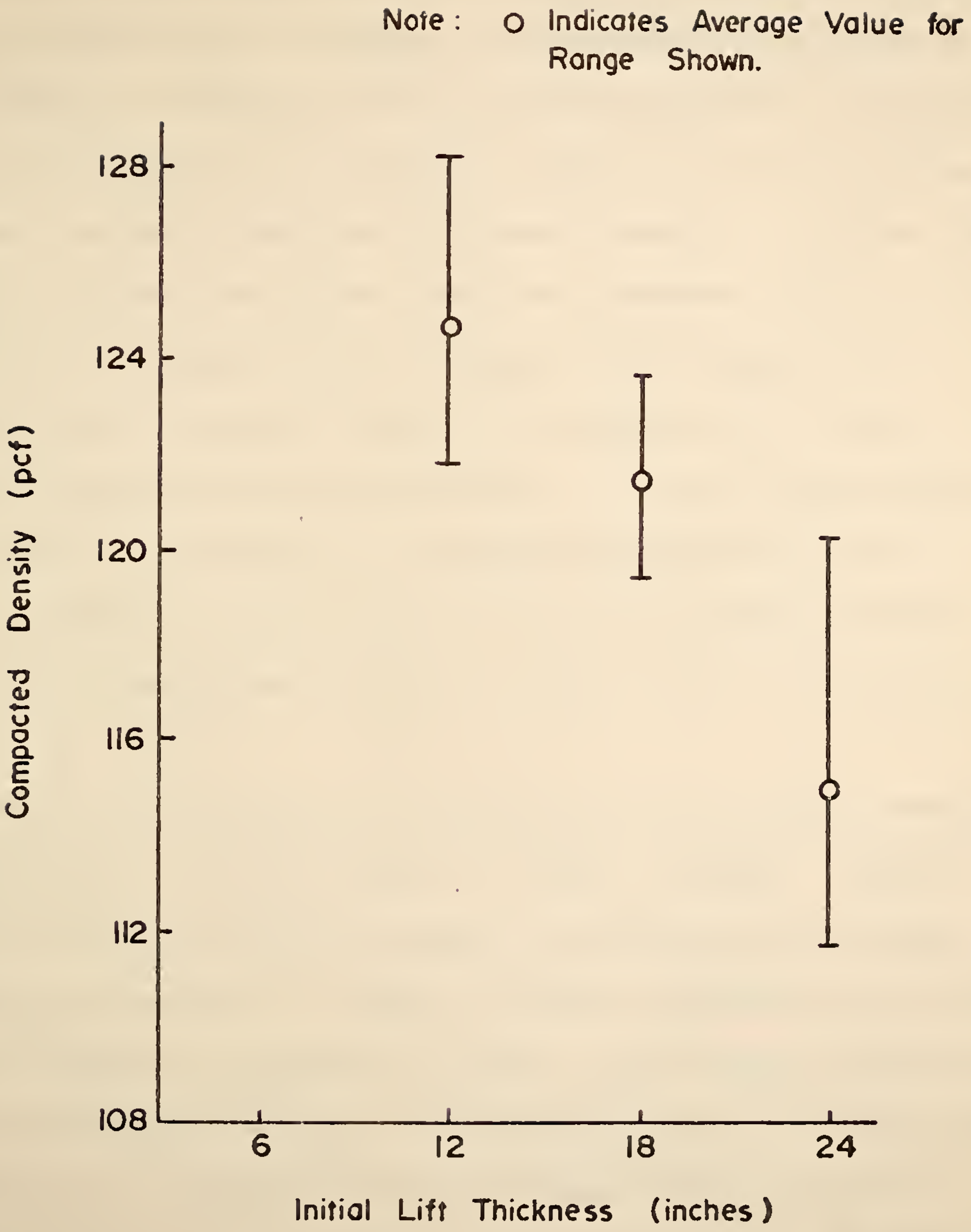


FIGURE 2 EFFECT OF INITIAL LIFT THICKNESS ON COMPACTED DENSITY FOR BORDEN SHALE. AFTER SCS (91).

revealed in the range of densities measured for each lift thickness increment. This range is shown by the vertical lines.

Figure 3 shows a fairly uniform increase in average unit weight as the compaction effort is increased. The information was obtained by the U. S. Bureau of Reclamation from a test pad constructed at Cachuma Dam (33). Here the field unit weights are represented by their difference (in pounds per cubic foot) from the standard laboratory condition. The vertical lines show the standard deviation in measured density for each effort level. It appears that the average density measured after fourteen passes is too low compared with the general trend. This illustrates the fact that measuring errors, as well as inhomogeneities in the material, can lead to an incorrect characterization of the material behavior. On any test pad, if only a few density tests are made, the data scatter and overlap may completely hide any meaningful trend.

The effect of increasing compaction effort on particle degradation can be expressed in the same manner as changes in density. Figure 4 shows the decrease in gradation modulus (corresponding to increased degradation as described in Chapter 3) before compaction and after one and three passes of a segmented pad roller. The gradation modulus was calculated from sieve analysis of bag samples taken from a single lift. The test pad was constructed with a soft clayey shale, by the ISHC. A similar trend is shown in Figure 5, which contrasts the degradation resulting from various effort combinations using two rollers, static and vibratory. This plot is based on averaged values from nineteen gradation samples taken by the ISHC (40).

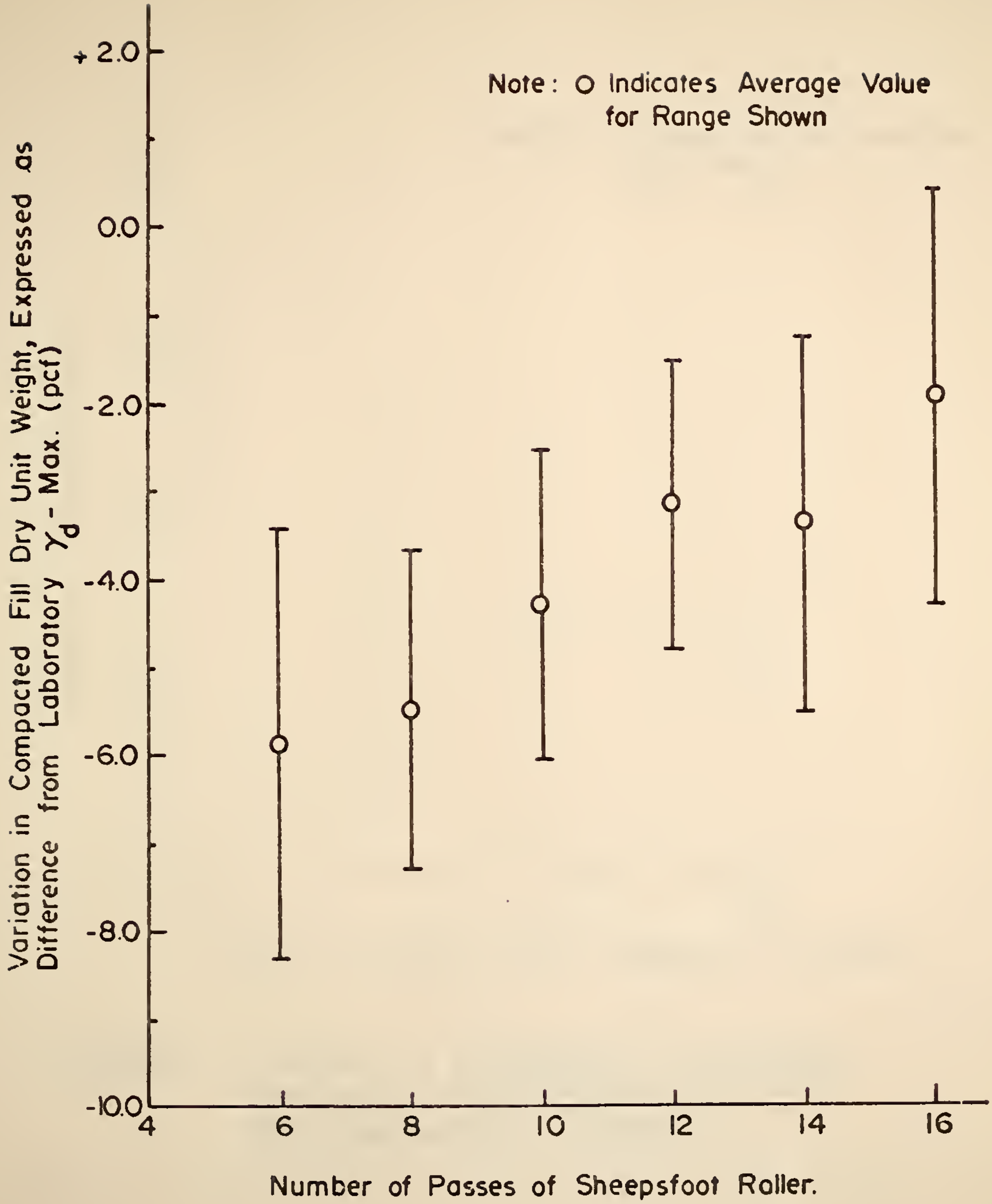


FIGURE 3 VARIATION IN DRY UNIT WEIGHT FOR INCREASING COMPACTION EFFORT. AFTER HILF (33).

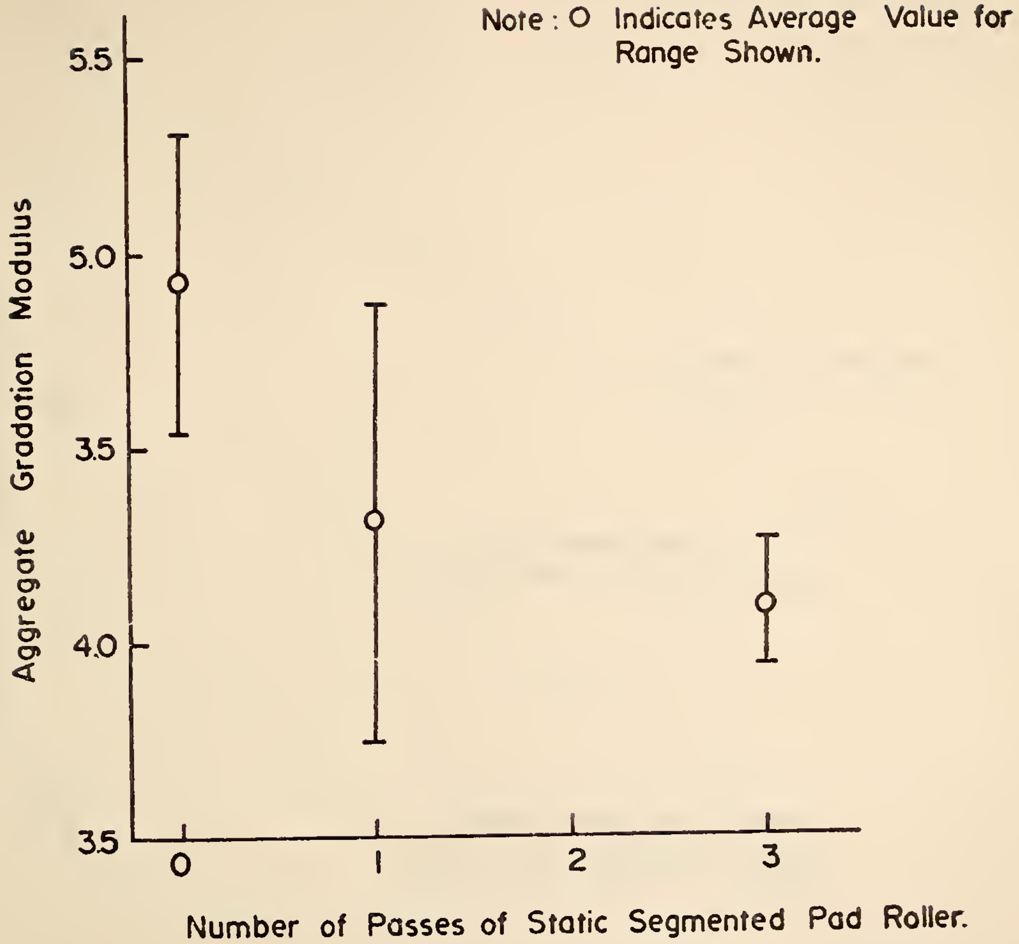


FIGURE 4 RANGE IN AGGREGATE GRADATION MODULUS FOR VARIOUS LEVELS OF COMPACTIVE EFFORT.

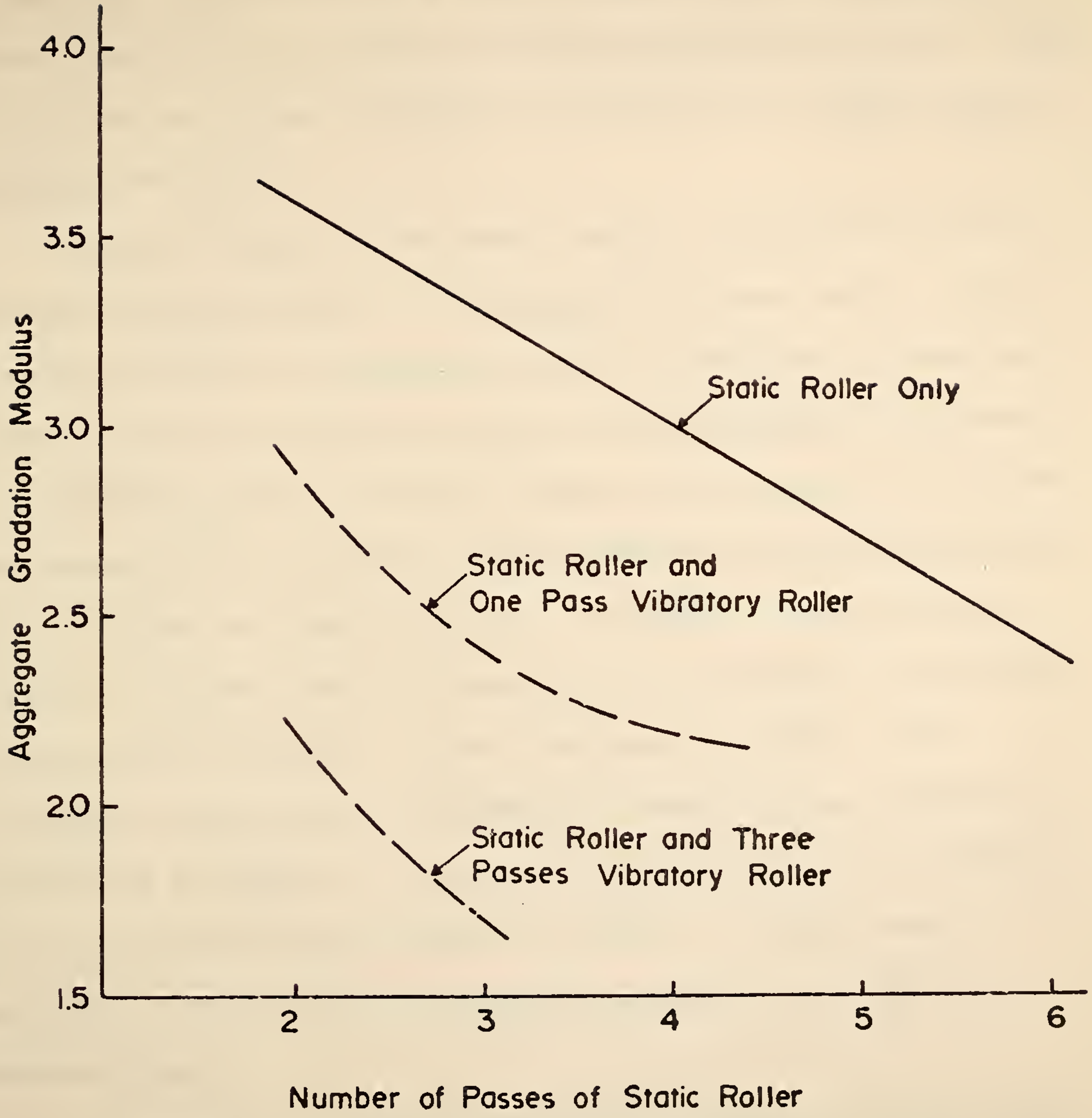


FIGURE 5 EFFECTS OF ROLLER USE ON AGGREGATE GRADATION MODULUS AFTER COMPACTION.

The effect of moisture content on shale degradation during compaction is shown in Figures 6 and 7, from a shale test pad built by ISHC (40). Figure 6 shows a fairly constant change in gradation modulus for two moisture ranges at different compactive efforts. Figure 7 highlights the effect of moisture on gradation for a single level of compactive effort only.

Although discussed in more detail later, it should be noted here that degradation parameters have relative significance only. There is no absolute measure of degradation unless the gradation is known both before and after the compaction process. Since gradation measurement is a "destructive test", requiring removal of the sample from the test pad, changes can be noted only between different samples taken from the test pad before and after compaction. To insure a valid comparison undisturbed by the random non-homogeneity of the material, sufficient samples must be available to allow statistical treatment. Thus comparisons of gradation, or degradation trends, are most valid when presented as in Figure 4. This statistical validity is not available for the test pad results in Figure 7, however, the trend shown is comparable to the results of controlled laboratory tests which are discussed later.

The degradation of carbonate aggregates in test pads using three types of roller has been investigated in other research at Purdue (8). Although direct correlation with the magnitude of degradation of shales is unrealistic, some of the results are worth noting. In comparing the compaction results for an uncrushed glacial gravel and three categories of crushed carbonate rock, the study noted that no single

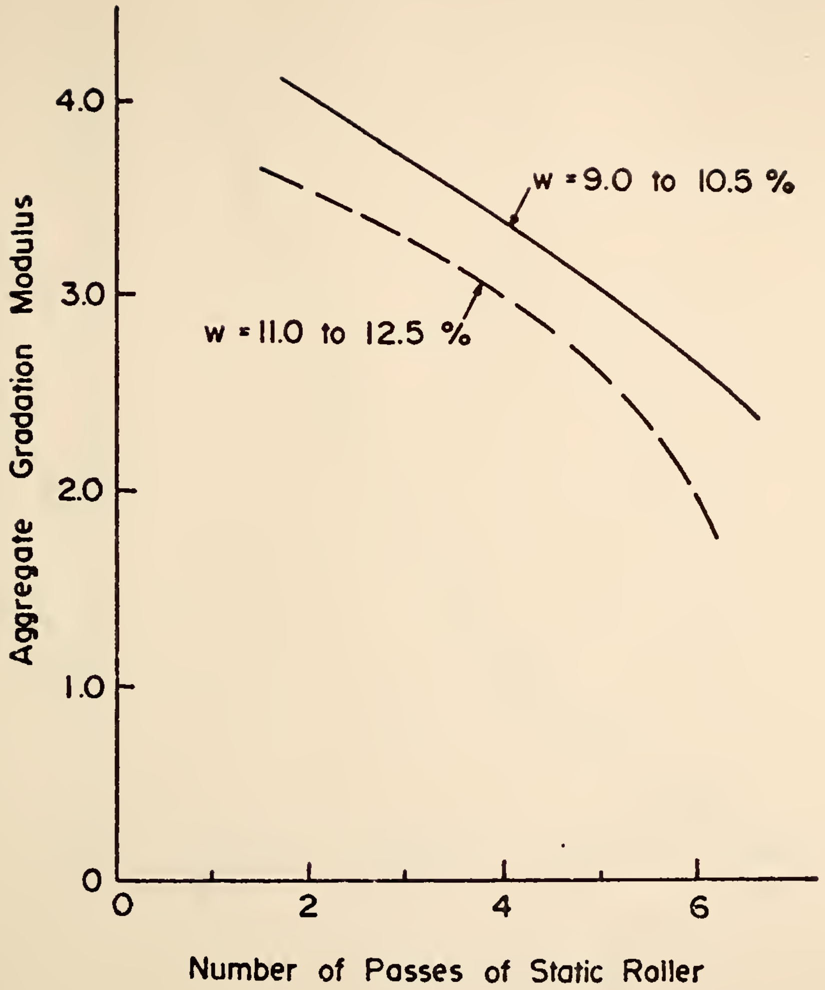


FIGURE 6 EFFECT OF WATER CONTENT ON AGGREGATE GRADATION MODULUS FOR INCREASING COMPACTION EFFORT.

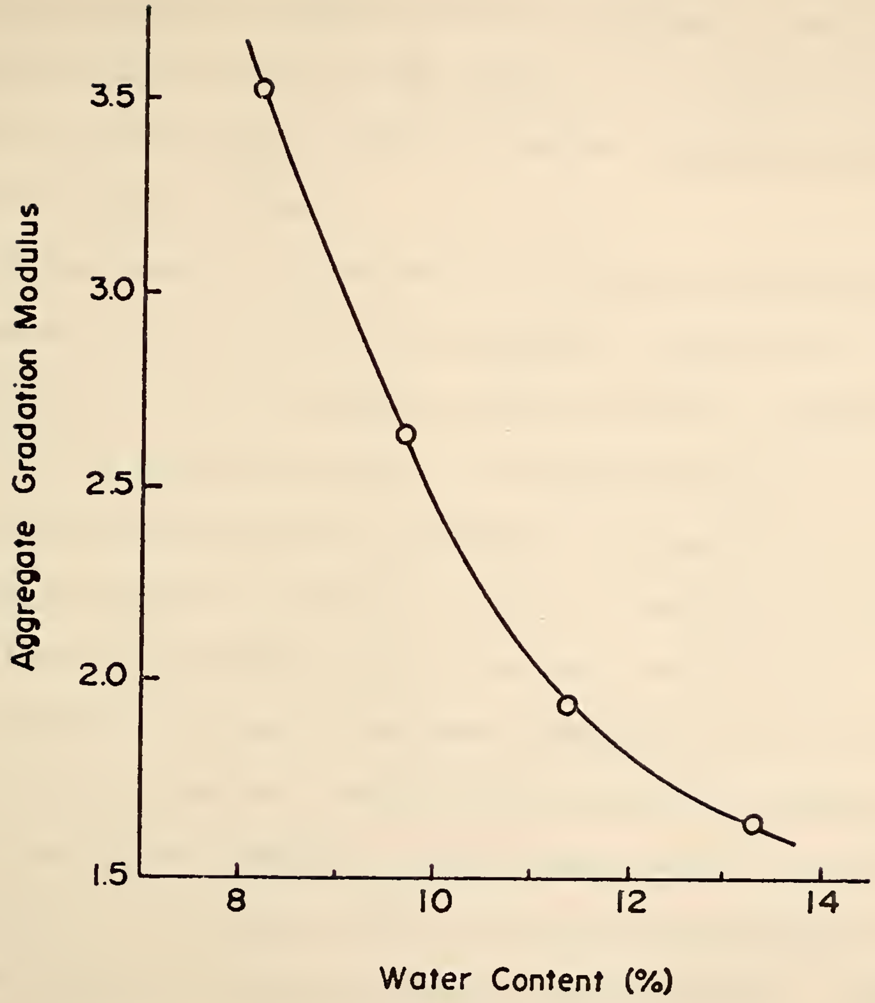


FIGURE 7 VARIATION IN AGGREGATE GRADATION MODULUS WITH WATER CONTENT, AFTER SIX PASSES OF STATIC ROLLER.

roller type (pneumatic, steel wheel, or vibratory) consistently produced the most or least degradation of the four different materials. The results also indicated that most degradation occurred during the first application of compactive effort. Subsequent roller passes produced relatively less particle breakage. This trend was especially evident for the vibratory roller, but varied in magnitude for the different materials tested.

The same study (8) investigated the effect of the compaction foundation and lift thickness on degradation. Compaction on a stiff foundation of reinforced concrete produced much more degradation than on a relatively yielding soil base. Increasing the loose lift thickness, but holding the effort constant, tended to decrease both density and particle degradation in the lift as a whole. Densification and degradation were most pronounced in the upper portion of each lift.

Results of the study on carbonate aggregates, as well as the results of shale test pads discussed above, show that accurate predictions of field compaction behavior can be made if sufficient data are available. The current state of the art for constructing shale embankments does not generally depend on such information. However, improvements in design criteria and cost reductions can be made through proper use of those test pad data.

2.3 Information From Shale Embankment Dams

Attempts have been made to apply the experience gained in the construction of shale dams to the construction of highway embankments (18, 74). In some cases there seems little reason for any comparison, as both the objectives and methods of the work differ. In other cases

however, the experience on dam jobs represents some of the best recorded information on shale construction (50, 74, 91).

Shale has been used in earth dams in two ways: as the main embankment material, or as a downstream mass zone only. In the first case, the objectives of construction are to create a uniformly densified, stable embankment. Although the loading scheme differs, the design and methods of construction are essentially the same for highway embankments. The shale must be placed and compacted uniformly to minimize internal settlements and loss of strength as the fill becomes saturated and slaking occurs. In addition, permeability considerations may require special care in achieving a dense gradation.

In the second case, the stability is dependent only on the mass properties of the shale with no particular attention to strength characteristics. The shale may or may not be compacted after placement. Its only purpose is to provide additional weight to increase the stability of the dam proper. Drainage may be provided, or the embankment slopes may be over-designed so that there is no strength problem so long as the total weight of the shale zone remains constant. In such a case, there is little comparison to highway work.

There are several other differences between the use of shale in dam and highway embankments. These differences occur in the areas of: initial investigation, location of borrow with respect to the fill, availability of equipment, magnitude of fill quantities, and service-life instrumentation and observation.

First of all, the geological exploration of the site and subsequent laboratory and field investigations of the borrow material are

very thorough for dam work. In contrast, the existence of shales to be used in highway construction may not even be anticipated until the contractor encounters the material in road cuts. Although some highway embankment designs are predicated by the known existence of large shale formations, the use of standardized fill sections by many highway agencies avoids the use of specialized investigations which could tailor design to specific material properties. In addition, the lead time from site or route selection to construction is more closely related to geotechnical factors in dam work.

Additionally, the source of borrow material is more restricted in highway work, where greatest economy is generally achieved through the balancing of cut and fill quantities. In dam construction, the fixed location of the construction site enables the designer to be more selective in using the local borrow. While many dams are designed around the use of material from "required excavation", selective borrowing may be more economical if the total quantity of material can be reduced through the use of better fill. This tends to stimulate more complete predesign investigation of the construction materials.

In the same way, better knowledge of the material and well established borrow locations can enable the economic use of special excavating and/or compacting equipment for dam construction. While highway construction with a variety of materials tends to produce compaction procedures for a general class of equipment, the literature includes several examples of "shale breakers" or specially fabricated rollers used on shale embankment dams (18, 53, 74). Also, special treatment of the shale, such as crushing or the addition of a lime

stabilizing agent, is easier with a fixed location, than with an advancing construction front.

Special problems result when only a relatively small amount of the fill is to be degradable shale. High mobilization costs or complete unavailability of specialized equipment requires the development of procedures using conventional equipment to handle the shale encountered in much highway work (40).

A final point of difference is that completed dam embankments are often the subject of special instrumentation and long term observation programs. These procedures are used in dam work because of the high risk of potential disaster should the embankment fail. Although there is only limited information regarding internal settlement observations for shale dams in the literature, this information has tremendous potential for a long term appraisal of the construction method used for any particular shale embankment.

No attempt will be made here to describe any specific dam construction. Examples of test pads associated with the construction of shale dams have already been given. Further information on shale dams can be found in the literature (18, 33, 50, 51, 53, 74, 84, 85, 89, 91, etc.).

2.4 Recommendations For Field Work

Based on observation of embankment construction by the Indiana State Highway Commission, personal interviews, and an extensive literature review, the following recommendations are made for increasing the density-degradation control of compacted shale embankments. These recommendations and observations are by no means a panacea, but are

intended to point out some possible areas for improvement.

For those shales which are resistant to mechanical breakage but have potential slaking and collapse problems, there is a great need for degrading the material before it reaches the fill. Breakage can be improved in several ways.

If the shale is to be excavated by ripping, the use of a multiple point ripper rather than the single tooth type can reduce the size of slabby pieces for some types of shale (15). Watering in the cut has been suggested to improve degradation. Possible treatment can include the use of sheepsfoot or spike tooth rollers in the cut, following ripping, to provide additional breakage before loading.

If the shale cannot be economically ripped, blasting can be used if attention is paid to the difficulty of controlling fragmentation. Closely spaced small diameter holes will improve fragmentation, as will the use of low velocity blasting agents or explosives with appropriate initiation delays (14, 87).

Excavation in benches, with the broken shale being pushed over the edge of the cut to a lower loading area will also increase breakage.

After the shale has been placed on the fill, the options for degrading it are somewhat reduced. Various combinations of static and vibratory rolling produce different amounts of degradation for different shales. Pug milling or the use of special break-up rollers have worked on some projects (74). One alternative to consider is compaction followed by disking to turn the material, and then final compaction rolling.

At this time, there is insufficient evidence to allow recommendation

of a particular equipment type or usage for all shales. Such recommendations are possible for particular shale deposits if field information from a test pad or previous project is available. The adequacy of test pads can be improved.

Generally, most test pads have been inadequately planned and attempt to investigate too many variables at once. Specifically, more tests (density, moisture content, and gradation) are needed for each lift. Isolated sampling for a large array of variables too often has resulted in confusing or incorrect interpretations. Proper guidelines are easy to apply: the test pad should consider only one independent variable at a time, with sufficient samples to allow statistical rejection of extreme values due to the inhomogeneity of the material and imprecise control of the compaction energy.

At the present time, no conclusion is possible regarding the relative merits of procedural or end-result compaction specifications for shales. Nor is it possible to state what the end result should be, except in very general terms. Such conclusions may be possible when more case histories describing the construction procedure and resultant density-degradation parameters are available, along with descriptions of the changes in the shale during the embankment's service life.

Each embankment constructed of shale can become a model for improving the construction of similar embankments in the future. Improved note-taking and an increased number of control tests can result in every shale fill being a "test pad" for future work. Under the present state of the art, such information is badly needed.

3. MEASUREMENT AND ANALYSIS OF PARTICLE DEGRADATION

3.1 Introduction

In the preceding chapter it was pointed out that successful use of shales in compacted embankments is dependent on controlling the strength loss and large internal settlements which can follow slaking. The problem is caused primarily by the slaked material collapsing into open voids within the fill. The troublesome aspects of this collapse can be reduced if the void size is controlled sufficiently. The size of voids within the embankment is a function of maximum particle size and particle size distribution. Proper embankment compaction serves to reduce the size of voids by both breaking up the shale pieces, and densifying the particle matrix. Some measure of particle degradation is needed if the effects of different compaction methods and equipment are to be observed and formulated. Degradation analysis can be an important tool in selecting the best compaction method for a particular shale.

3.2 Gradation Measurements

Shale particles break up during compaction through the action of three basic mechanisms: fracturing, abrasion, and moisture-induced effects. The preceding chapter discussed the various degrading influences that are present during different embankment construction stages. Regardless of the specific causes or mechanisms of particle

breakdown, some measure of the particle size distribution or gradation is needed to represent both the initial and final condition of the shale. Such distributions are commonly treated as either (a) cumulative or (b) differential frequency distributions and are an accepted part of conventional soil mechanics practice (10, 80, 81). Less commonly, particle shape may be of special importance in describing the gradation of a material, and tests have been developed using this criteria also (11, 12).

In the following discussion, several methods of describing the gradation of shale samples are considered. All are based on some arithmetic manipulation of the percent by weight of shale particles which were retained on a set of U. S. Standard sieves. To measure the breakage of particles during any type of test, each sample was sieved both before and after testing. Specific details of the sieving operation are discussed in Appendix B.

Taken alone, the results of sieving tests are not an adequate means of assessing particle degradation. The data obtained from sieving consist of a series of paired numbers: each sieve mesh size and a corresponding fraction of the sample. Figure 8 is a conventional cumulative plot of the before and after gradation curves for a sample of Shale No. 11 which was subjected to kneading compaction. This graphic display is a good method of illustrating the initial and final gradations of a single sample, however the display is limited to a fairly small amount of data. Figure 8 would become quite cluttered if a variety of initial and final sample gradations were shown for several different magnitudes of degradation, i.e. for several levels of

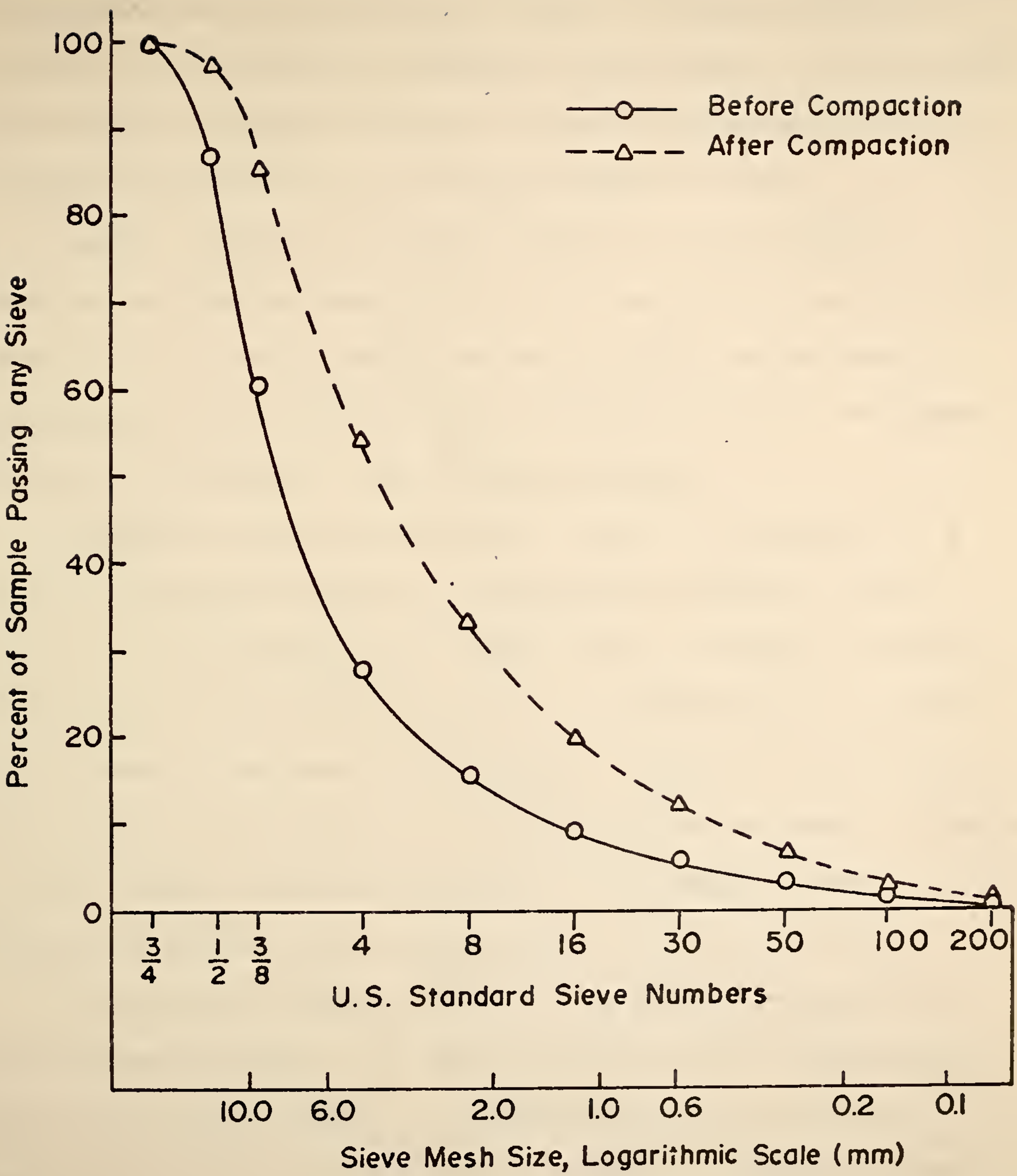


FIGURE 8 GRADATION CURVES FOR A SAMPLE OF SHALE NO. 11 SUBJECTED TO KNEADING COMPACTION.

compactive effort. Furthermore the series of paired coordinates which define each gradation curve is not amenable to simple statistical treatment. In comparing the gradation of many samples, a single number representing each sample gradation is much more convenient to use in regression analyses than is a series of numbers for each.

Several terms have been used to provide a single number to describe the shape and range of the grain size distribution curve, or gradation, of a sample. Such terms are divisible into two general categories: gradation coefficients and gradation indices. Both types of terms are described in the following sections.

Regardless of the method chosen to describe the gradation of a sample, degradation analysis of shale materials remains an inexact process. The nonhomogeneity of shales results in ranges, rather than single values, of material parameters. The degradation of a specific type of shale in response to some specific compactive effort may vary widely due to the variability of the material from one sample to another. For this reason, and those discussed in the preceding chapter, degradation analysis must be approached as a statistical process.

In laboratory compaction tests it is possible to control the initial sample gradation. In the field such control is impossible, and because sieve analysis is a "destructive" test, an average of many sample gradations must be used to compare the before and after conditions of the material. A sieve analysis determines what amount of material is of certain sizes only, it does not indicate the changes undergone. A "large" particle has to undergo more breakage than does a "small" particle before both became fragments of a certain size.

Moavenzadeh (58) concluded that sieve analyses are adequate only for studying the pattern of degradation, while a true measure of degradation must be based on the percent change in some sample parameter. Such a measure of degradation would include terms representing both the initial and final sample gradations, thus eliminating the need for controlling the initial condition. Moavenzadeh's work will be discussed further, in the Gradation Indices section.

3.3 Gradation Coefficients

Several gradation coefficients have been developed to represent soil grain size distribution curves for problems in soil mechanics, hydrology and geology (10, 80, 81, 82). Values for the several coefficients discussed below can be obtained directly from sieve test data which have been plotted in gradation curves as shown in Figure 8. Gradation coefficients are intended to describe the shape and/or position of gradation curves for different samples. The coefficients enable comparisons of several sample gradations without requiring that they be plotted on the same graph. Gradation coefficients also provide a "number" to be used as a variable in mathematical expressions intended to compare gradation with other sample parameters.

Most engineering properties of granular soils or fragmented rock are not directly dependent on particle size (81, 82). For this reason the physical meaning of the various gradation coefficients has not been extensively investigated or documented.

All of the gradation coefficients are based on obtaining the maximum particle size which represents some fraction of the total sample. The most commonly used gradation coefficient is the "effective

size", originally developed by A. Hazen in 1892 (82). This coefficient, symbolized as D_{10}, was found by Hazen to be proportional to the permeability of granular soils. The notation, D_{10}, represents the maximum particle size which exceeds 10% of the sample grains, but is smaller than 90% of the sample. D_{10} is essentially a measure of the coarseness of the smallest 10% of the particles in a sample (82).

Hazen also developed the "coefficient of uniformity", C_u, as an indicator of permeability. This coefficient is defined as:

$$C_u = \frac{D_{60}}{D_{10}}$$

where D_{60} represents the maximum particle size of the smallest 60% of sample particles, and D_{10} is defined as before. The coefficient of uniformity is an indicator of the range of particle sizes in any particular sample, but is independent of the magnitude of these sizes. A large value of C_u indicates that the sizes D_{60} and D_{10} differ substantially, a value of C_u approaching 1.0 indicates the particles are more uniformly sized. Sowers and Sowers (80) have suggested that $C_u < 4$ indicates uniform soils and that $C_u > 6$ indicates well-graded soils. Terzaghi and Peck (82) have questioned the value of using the ratio of D_{60} and D_{10}, as opposed to some other ratio of relative sizes, for sample characterization.

A third gradation coefficient, the "coefficient of concavity", C_c, has been used to describe the shape of grain size distribution curves (10). This coefficient is defined as:

$$C_c = \frac{D_{30}^2}{(D_{10})(D_{60})}$$

The value of C_c approaches unity for well graded or uniform soils. A high value of C_c indicates the sample is gap-graded or sorted into two or more narrow size ranges (10).

Another gradation coefficient which has been used is the "coefficient of sorting" (80). The coefficient of sorting, S_o, has been proposed for geologic classification work, and is defined as:

$$S_o = \left(\frac{D_{75}}{D_{25}}\right)^{1/2}$$

Any of the various gradation coefficients may be used to describe the degradation of shale particles during compaction. Using Hazen's coefficient of uniformity for example, the degradation of shale samples subjected to uniform compactive effort could be described as:

$$\text{DEGRADATION} = \frac{(\sum C_u)_a}{(\sum C_u)_b} \left(\frac{d}{c}\right)$$

where the subscripts b and a denote the sums of the before and after compaction uniformity coefficients, respectively. The term c represents the number of "before" samples seived, and d is the number of samples sieved after compaction. Alternatively, the arithmetic differences between the average "before" and "after" uniformity coefficients might be used to represent the degradation due to compaction. In either case an average of several samples is needed to

represent both initial and final conditions because of the non-homogeneous nature of the material, which may appear as variations in gradation between supposedly similar samples.

If the degradation resulting from different levels of compactive effort is to be analyzed, the percent difference between initial and final average gradation coefficients should be used. As discussed by Moavenzadeh (58) and Aughenbaugh et. al. (8), the percent difference is a more precise indicator of the actual change in the material than is an arithmetic difference.

Gradation coefficients in general have one major disadvantage compared to the gradation indices discussed in the next section. When considering the degradation of compacted shales, the change in gradation for the entire sample is of interest. None of the gradation coefficients indicates more than the maximum size of two or three fractions within the sample. Using the coefficient of uniformity as an example, Figure 9 shows the gradation curves for two shale samples having very different gradations but the same value of C_u.

The dashed curve in Figure 9 could represent the after-compaction condition of a sample which had the initial gradation described by the solid curve. In this case, a degradation parameter based on the change in C_u would falsely indicate that no change in sample gradation had occured. Similar cases of misleading degradation parameters could be formulated using any of the gradation coefficients.

3.4 Gradation Indices

Gradation indices differ from gradation coefficients in that the indices utilize a broader range of information to describe a sample.

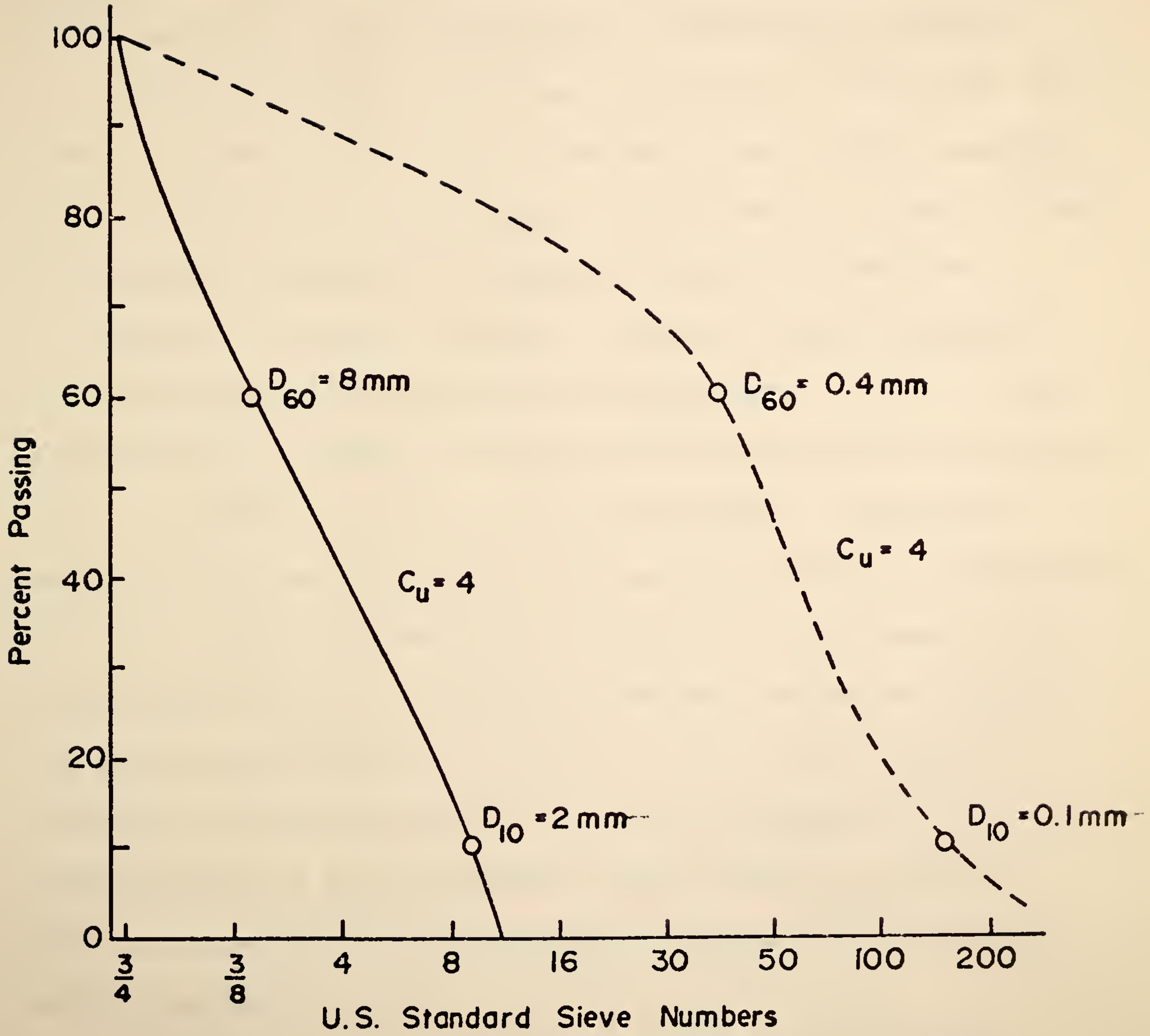


FIGURE 9 GRADATION CURVES FOR TWO SAMPLES WITH THE SAME COEFFICIENT OF UNIFORMITY.

Rather than measure the maximum size of some sample fraction(s), the gradation indices indicate the percentage of a sample included in each of several particle size groups. The individual size groups used in determining a gradation index are contiguous over the entire particle size range of the sample. For this reason gradation indices are much less susceptible to the kind of ambiguity demonstrated in Figure 9.

Two kinds of gradation index were investigated in this study. The index of crushing (IC) and aggregate gradation modulus (AGM) were both found useful in describing the change in gradation of the samples tested, and in relating degradation to other shale compaction parameters.

The use of the index of crushing to measure particle degradation due to compaction has been described by Aughenbaugh et al. (8). This gradation index is defined by the sum of the weighted fractions included in each of several sieve groups. This investigation used a smaller range in size for defining the sieve groups than was used by Aughanbaugh et al. (8). Each sample fraction is weighted by a factor equal to the mean equivalent mesh size of the two sieves which define the particle size group being evaluated. The value of IC for a given sample is computed as the percent change in the sums of the weighted fractions obtained before and after degradation. Table 3 shows a sample data sheet, the sieve groups, and the method of calculating IC, that were used in this study.

The aggregate gradation modulus also uses size group factors based on an effective diameter of representative particle sizes (37). The effective diameter, $\bar{d}$, is defined as:

Table 3.

Data Sheet And Method of Calculating IC, After Aughenbaugh et al. (8).

Size Group Definitive Sieve Numbers	Definitive Sieve Mesh (mm)	Mean Equivalent Mesh Size (mm)	Percent of Sample Included	Weighted Fraction of Sample
3/4 to 1/2	19.1 to 12.7	15.9	A_1	$15.9 \times A_1$
1/2 to 3/8	12.7 to 9.52	11.1	A_2	$11.1 \times A_2$
3/8 to #4	9.52 to 4.76	7.14	A_3	$7.14 \times A_3$
#4 to #8	4.76 to 2.38	3.57	A_4	$3.57 \times A_4$
#8 to #16	2.38 to 1.19	1.78	A_5	$1.78 \times A_5$
#16 to #30	1.19 to 0.59	0.89	A_6	$0.89 \times A_6$
#30 to #50	0.59 to .297	.444	A_7	$.444 \times A_7$
#50 to #100	0.297 to 0.149	.223	A_8	$.223 \times A_8$
#100 to #200	0.149 to 0.074	.112	A_9	$.112 \times A_9$
#200 to pan	0.074 to 0.000	assumed as 0.037	A_{10}	$.037 \times A_{10}$
				$B = \Sigma A_i$

B_0 = Sum of Weighted Fractions Before Degradation

B_1 = Sum of Weighted Fractions After Degradation

$$\text{Degradation} = \text{IC} = \frac{B_0 - B_1}{B_0} \times 100\%$$

$$\bar{d} = \frac{0.4343\ (d_1 - d_2)}{\log_{10}\ (d_1/d_2)}$$

where d_1 and d_2 are the mesh openings, in millimeters, of the sieves which define the limits of the particle size group. From the effective diameter, a modulus term, A_i, for each particle size is determined as

$$A_i = \frac{\log_{10}\ (54.8) - \log_{10}\ (\bar{d}_i)}{\log_{10}(2)}$$

for each sieve group "i". A_i is the factor used to correct the percentage of the sample which is included in each particle size group. The gradation index, AGM, is the sum of all the A_i corrected terms for a sample. Degradation is measured by the arithmetic or percent change in AGM.

For particle degradation analyses, the final sample gradation will consist of more fines than were present in the initial condition. This means that the value of the aggregate gradation modulus will increase for any sample that is undergoing degradation, and the arithmetic and percent changes in AGM will have values less than zero. In order to eliminate negative values from the statistical regression analyses used later in this report, the following notation has been adopted.

$$\begin{aligned} AA &= \text{arithmetic change in AGM} \\ &= (AGM_i - AGM_f)\ (-1) \end{aligned}$$

$$\begin{aligned} PA &= \text{percent change in AGM} \\ &= \frac{AGM_i - AGM_f}{AGM_i}\ (100\%)\ (-1) \end{aligned}$$

Thus for the purposes of this report, an increase in the value of aggregate gradation modulus will henceforth be considered to indicate increased particle degradation.

Table 4 compares the AGM and IC correction factors obtained for the same particle size groups. Table 4 shows clearly the major difference between the factors which determine these two indices. As the mesh sizes describing the particle size groups decrease, so do the IC group weighting factors. In contrast, the AGM group weighting factors show an increase in magnitude as the group size decreases. This means that the AGM index is more sensitive to changes in the smaller particles within a sample, i.e. the production of fines, than is the IC.

Both the IC and AGM can be used to determine and compare the degradation of samples with dissimilar initial gradations. This is because the weighting factors for both indices are based on real measures of sieve openings which retain or pass the particles of a sample. When degradation is expressed as the percent change in either gradation index, thus relating both initial and final conditions, the "real" base of the weighting factors allows direct comparison of samples without the need for scaling factors or oversize corrections. This enables degradation comparisons between small scale laboratory tests and actual field compaction.

3.5 Other Measures of Gradation

Moavenzadeh (58) has described a gradation index based on assumed particle surface area, similar to the AGM. Using idealized geometrical shapes (cubes, spheres, and parallelepipeds), Moavenzadeh calculated a

Table 4.

Comparison of IC And AGM Group Weighting Factors.

Sieve Numbers Defining Particle Size Group	Index of Crushing Group Weighting Factors	Aggregate Gradation Modulus Group Weighting Factors
3/4 to 1/2	15.9	1.80
1/2 to 3/8	11.1	2.31
3/8 to #4	7.14	3.00
#4 to #8	3.57	4.00
#8 to #16	1.78	5.00
#16 to #30	.890	6.00
#30 to #50	.444	7.00
#50 to #100	.223	8.00
#100 to #200	.112	9.00
#200 to pan	.037 (assumed)	10.00 (assumed)

surface area weighting factor for each of ten particle size groups. The validity of Moavenzadeh's particle shape assumptions vs. the assumptions used to determine the AGM group weighting factors were not investigated in this study. Moavenzadeh also describes the use of a specific gravity correction factor for comparing the gradation of different materials.

Davies and Stewart (21) have described the use of the Bagnold grading parameters to describe the gradation of sands. The Bagnold parameters are four variables which describe the slope and position of a particle gradation curve plotted as the logarithm of the percent retained on each sieve vs. the logarithm of particle size diameter. The particle size diameters are obtained from sieve tests. Initial attempts to use the Bagnold parameters to describe the change in gradation of shales due to long-term soaking degradation indicated the parameter values were subject to questionable interpretation due to scatter in the data. No attempt was made to investigate the applicability of the Bagnold parameters for degradation of shales due to compaction. The results reported by Davies and Stewart (21) suggest that further analysis might be worthwhile.

4. COMPACTION-DEGRADATION TESTS

4.1 Kneading Compaction

4.1.1 Introduction and Description of Apparatus

Kneading compaction tests are intended to model actual field compaction conditions. Other investigators have used this type of test to simulate the action of both pneumatic compactors (29, 64) and sheepsfoot compactors (92).

Kneading compaction differs from static and impact compaction in that the compactive effort is applied through cycles of gradually increasing and decreasing load. This action is comparable to the load-deformation reaction that occurs under the wheels of static compactors in the field (92). The name "kneading compaction" is taken from the movement of particles within the soil or rock sample being compacted. This movement occurs as a kneading or shoving action imparted to the soil particles by the rotation of the drum or tires of field compactors, or by the compacting foot in the laboratory test.

The kneading compaction apparatus used in this investigation is an electrically driven semi-automatic device, and has been described in detail by Aughenbaugh et al. (8) and by Gaudette (29). The apparatus can be used to compact four or six inch diameter samples, up to about twelve inches in height. The tests reported herein were conducted on four inch diameter samples with a maximum height of five inches for the loose shale before compaction.

The kneading compaction apparatus applies compactive effort to the sample through a two inch diameter segmented-circle ram or compaction foot. The compaction sample is rotated 60° in the horizontal plane between applications of load. The load, or pressure exerted on the sample by the compaction foot, can be varied through a pneumatic-hydraulic control system. This allows the investigator to select a compaction effort level comparable to the contact pressure exerted by various types of field equipment. The constant pressure control allows the compaction foot to "walk out" of the sample as it densifies.

Compaction effort is divided into four distinct stages for each load application cycle of the kneading compaction apparatus.

1. The compaction foot descends to the sample surface and pushes into the sample.
2. Resistance of the sample to the foot penetration effected primarily through lateral restraint, causes the ram pressure to build up to a pre-selected level.
3. The maximum ram pressure is maintained for a finite period of time, and then begins to decrease.
4. The ram pressure decreases to zero, the compaction foot is raised from the sample surface, and the sample then rotates to a new position.

A time-pressure curve representing the action described in the second and third stages is shown in Figure 10.

The application of compactive effort described above causes both vertical and lateral stresses and strains within the sample material. According to Wahls et al. (92) the shear stresses and strains developed

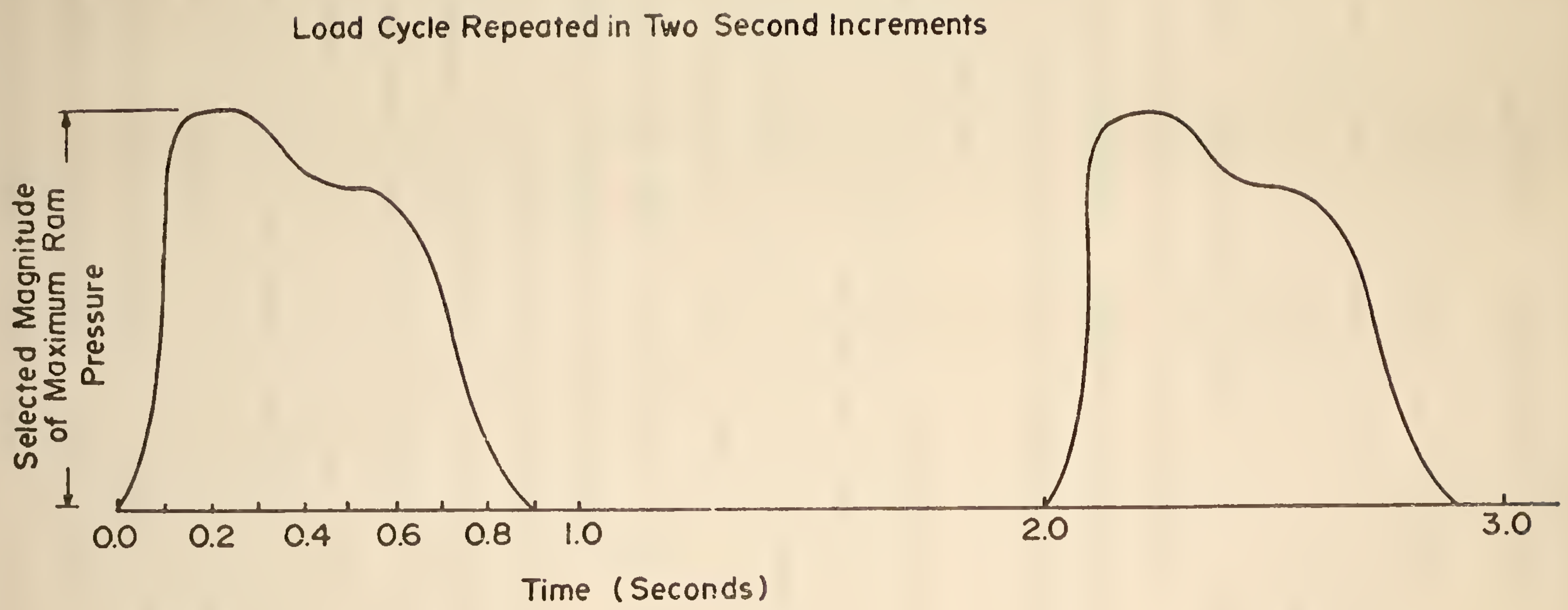


FIGURE 10 TIME-PRESSURE CURVE FOR KNEADING COMPACTION TEST APPARATUS. AFTER AUGHENBAUGH et al. (8).

during kneading compaction "...appear to be analogous to those developed during compaction by a sheepsfoot roller."

4.1.2 Test Description and Procedure

Kneading compaction tests were performed on samples of Shale No. 11, the Attica model shale, to investigate the relation between compaction effort, compacted sample density (unit weight) and particle degradation. The compaction effort was considered to vary with both the number of load applications and the maximum foot pressure. Tests were performed using twelve levels of compaction effort for each of three groups of samples.

Three magnitudes of compaction foot pressure were used for the tests, 50, 100 and 200 psi. Samples were subjected to 50, 100, 200 and 500 load applications at each of the three pressures. From this it was hoped to determine the relative effects of increasing the compaction roller weight and/or increasing the number of roller passes on both densifying and degrading the model shale.

A review of the literature indicated there is no standard method of test for kneading compaction of soils using a semi-automatic apparatus. Test procedures have been developed for kneading compaction of asphaltic materials (8, 29), but these were found to be unsuitable for this study. The test procedures described by Aughenbaugh et al. (8) were found to be excessively time consuming (twenty layer method) or to cause non-uniform densification and degradation within the compacted sample (one layer method). After initial trials with the Attica shale, a test procedure was developed for use in this investigation, as described below.

A sample of crushed shale weighing approximately two pounds was placed in a plastic bag and gently kneaded by hand to insure mixing of fine and coarse particles. The sample was then poured out onto a large sheet of paper, in the form of a circular mound. The sample was split into three parts of approximately equal size, using a small spatula. One third of the sample was loosely poured into the 4 inch diameter cylindrical steel mold, and compaction was started. The number of load applications was counted. When 20% of the total number of load applications selected for the test had been completed, the second third of the sample was gradually poured into the mold. When 30% of the load applications were completed, the final third of the sample was gradually added. The final 60% of the total number of load repetitions was thus applied to the entire sample, as shown in Figure 11. Note that the second and third portions of the sample were added during the compaction process. The action of the compaction ram and the mold rotation allowed an even distribution of the shale particles as they were poured in. While this procedure is not directly comparable to field compaction, it did eliminate gross stratification (of different sized particles) within the samples, as observed at the end of the tests.

Three different sample gradations of Shale No. 11 were tested. Of primary interest is the "A" gradation which was used in all of the compaction tests described in this report. This sample gradation consisted of all particles passing the 3/4 inch sieve, from material which had passed through a jaw crusher. Preparation of compaction samples is described in Appendix B, General Laboratory Procedures.

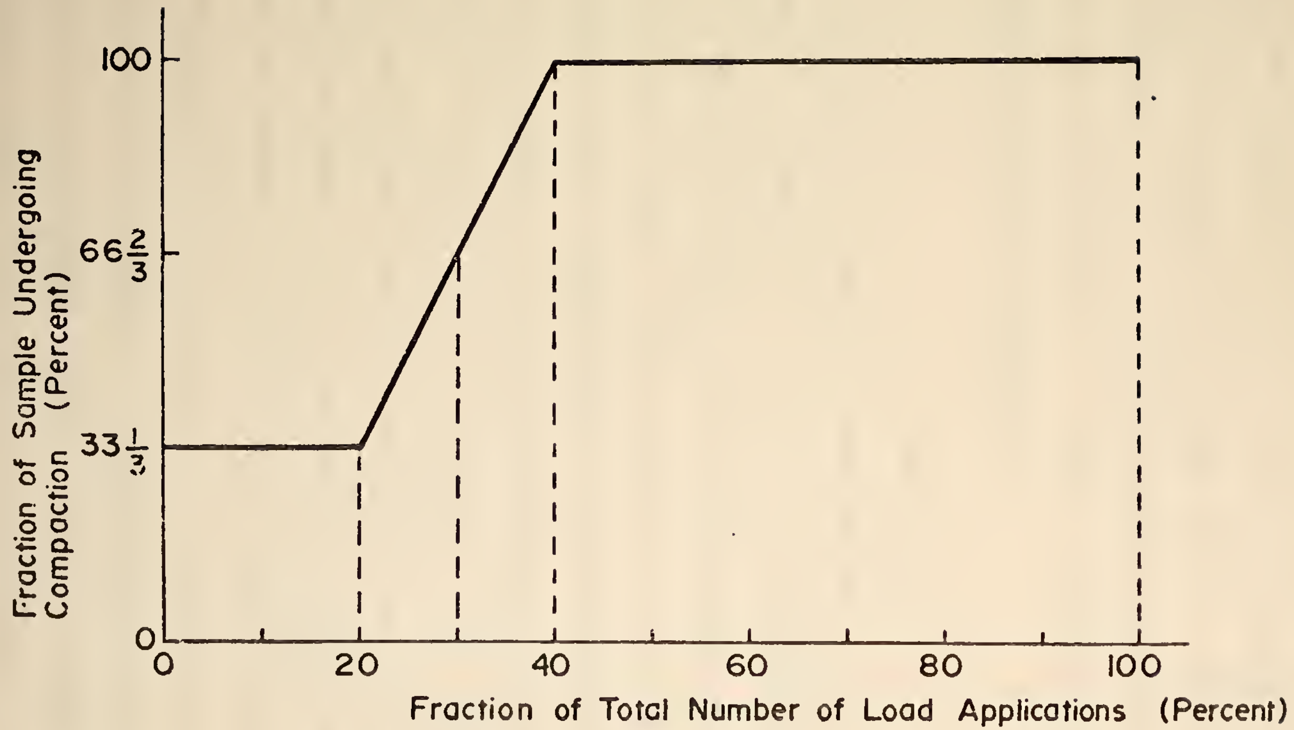


FIGURE 11 DIAGRAM OF KNEADING COMPACTION SAMPLE FEED SEQUENCE.

Since the "A" gradation was used for all of the compaction tests described in this report, the results of these tests can be compared directly, without using any mold-particle size correction factor.

Kneading compaction tests were also performed on two other sample gradations, labeled "B" and "C". The "B" gradation consisted of shale particles which passed the 3/4 inch sieve but were retained on the 3/8 inch sieve. The "C" gradation consisted of shale particles between the 3/8 inch and No. 8 sieves. These three initial sample gradations are shown in Figure 12.

4.1.3 Test Results

The results of kneading compaction tests on Shale No. 11 are tabulated in Appendix D. In general it was found that repetitions of kneading compaction tests with all conditions held constant did not yield the same results. Test results for all three initial sample gradations include anomalous data which cannot be excluded from the analyses on basis of observations made during or immediately after the compaction tests. With a few exceptions, tests performed on all three initial sample gradations were performed twice for each effort level. Although this enabled the performance of a large number of tests to compare the effects of gradation over a large compaction effort range, the resulting data are not amenable to frequency analysis for any single set of test conditions. The following results are thus discussed in terms of mean values in order to eliminate some of the discrepancies noted in the individual test results. Experience gained from the analysis of the kneading compaction tests was applied to

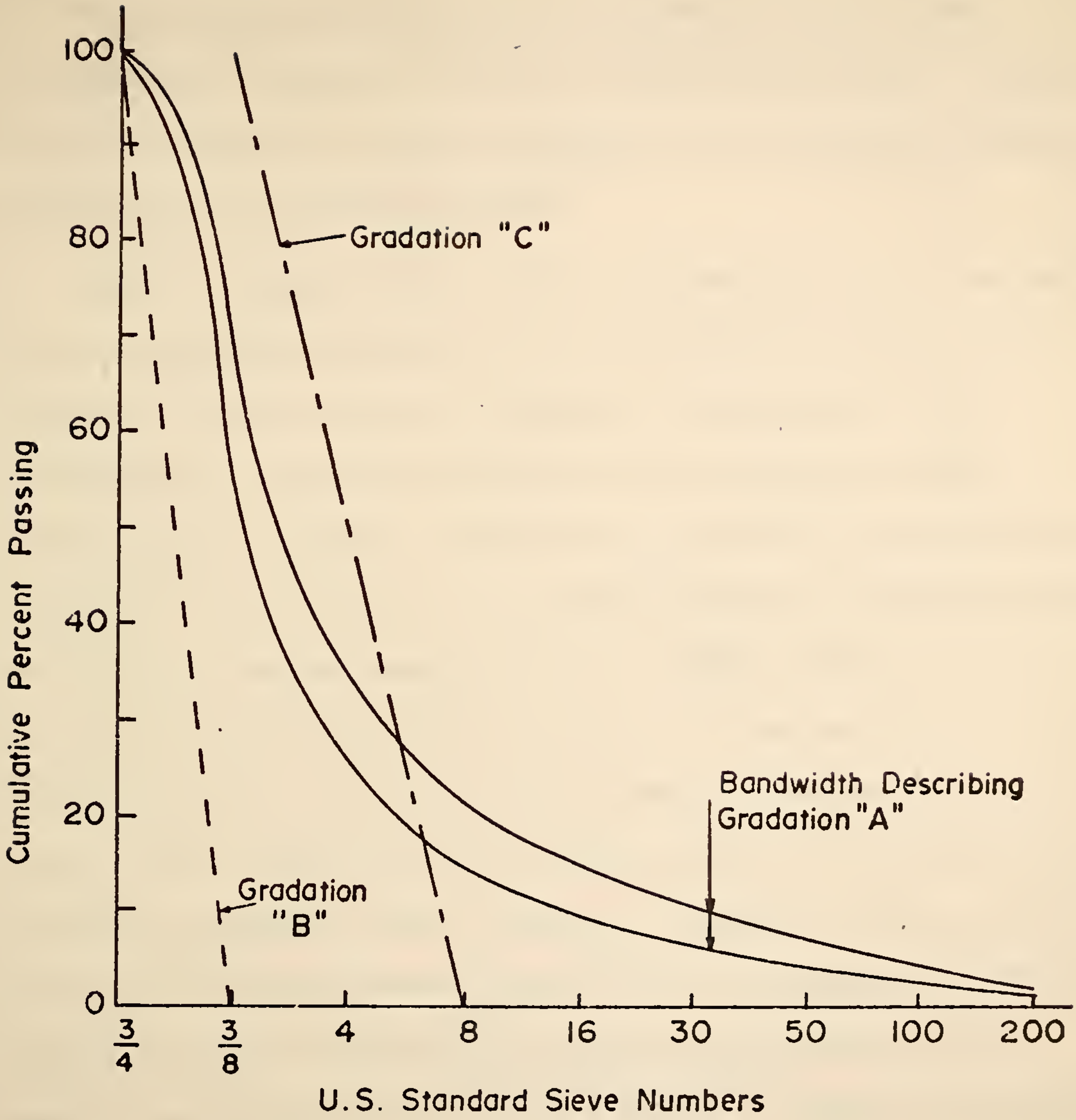


FIGURE 12 COMPARISON OF INITIAL SAMPLE GRADATIONS USED IN KNEADING COMPACTION TESTS.

improving the design of other laboratory tests for the same material, in this investigation.

The kneading compaction process resulted in a fairly good correlation of compaction effort with sample densification. Figures 13 and 14 show families of curves developed for the compaction of samples with the type "A" initial gradation.

From the averaged results shown in Figures 13 and 14 it can be seen that the compacted unit weight is more sensitive to changes in the compaction foot pressure then to changes in the number of load applications, although the magnitude of this sensitivity varies considerably. In general it was found that doubling the magnitude of compaction foot pressure for some constant number of load applications resulted in an average sample moist unit weight increase of 7.2 pcf. Doubling the number of load applications for a constant level of foot pressure resulted in an average sample moist unit weight increase of only 4.0 pcf. This difference in effect appears even more significant when either compaction effort variable is changed by a factor greater than two. An effort increase factor of four, for example, results in average unit weight increases of 14.4 pcf for the change in foot pressure, as opposed to 6.5 pcf for a comparable change in the number of load applications.

Figure 15 shows the average compacted unit weights resulting from kneading compaction tests on samples with the "B" and "C" initial gradations. For the higher levels of compactive effort, most of the tests resulted in greater values of compacted unit weight for the "B" gradation samples, than for the "C" samples. This tendency is not

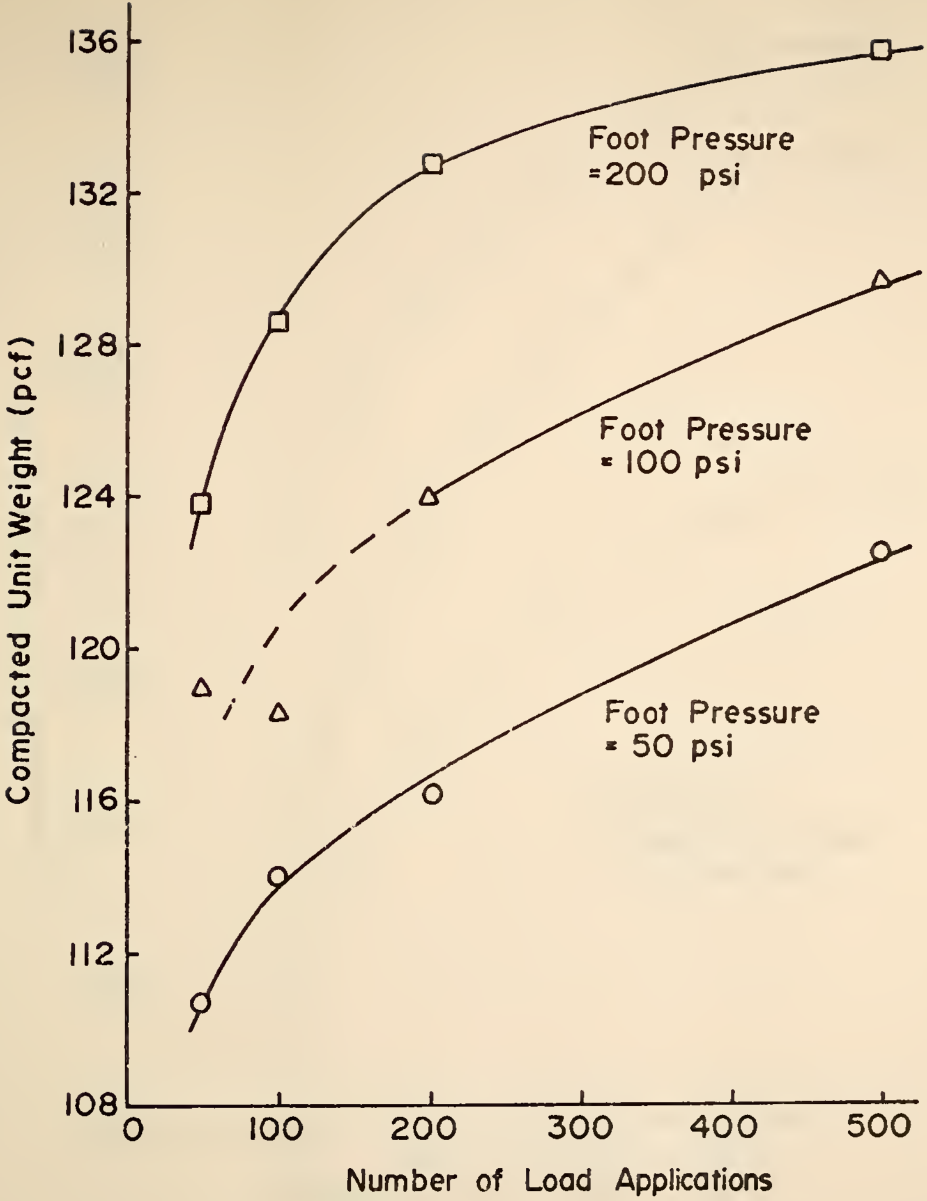


FIGURE 13 RELATION BETWEEN AVERAGE COMPACTED UNIT WEIGHT AND NUMBER OF KNEADING COMPACTION LOAD APPLICTIONS FOR DIFFERENT MAGNITUDES OF FOOT PRESSURE.

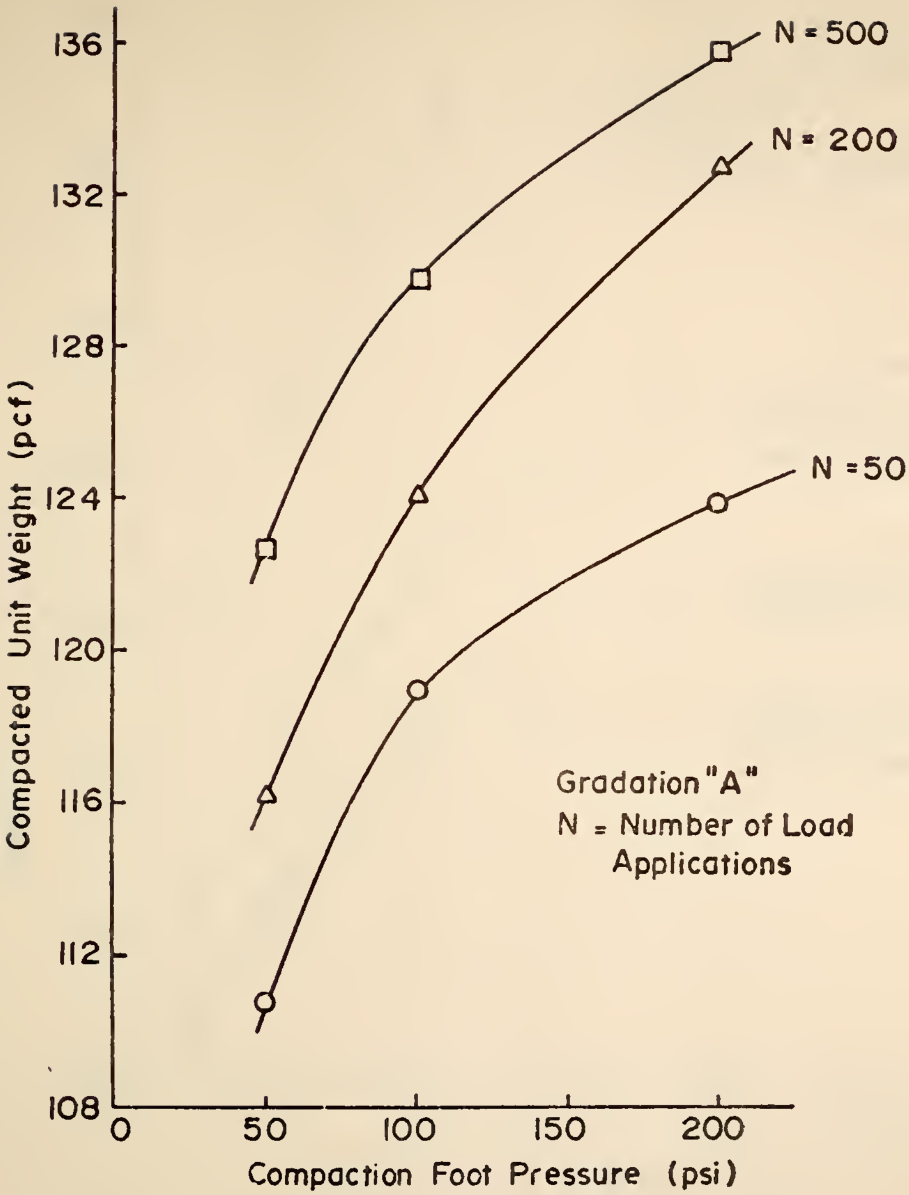


FIGURE 14 RELATION BETWEEN AVERAGE COMPACTED UNIT WEIGHT AND KNEADING COMPACTION FOOT PRESSURE FOR DIFFERENT NUMBERS OF LOAD APPLICATIONS.

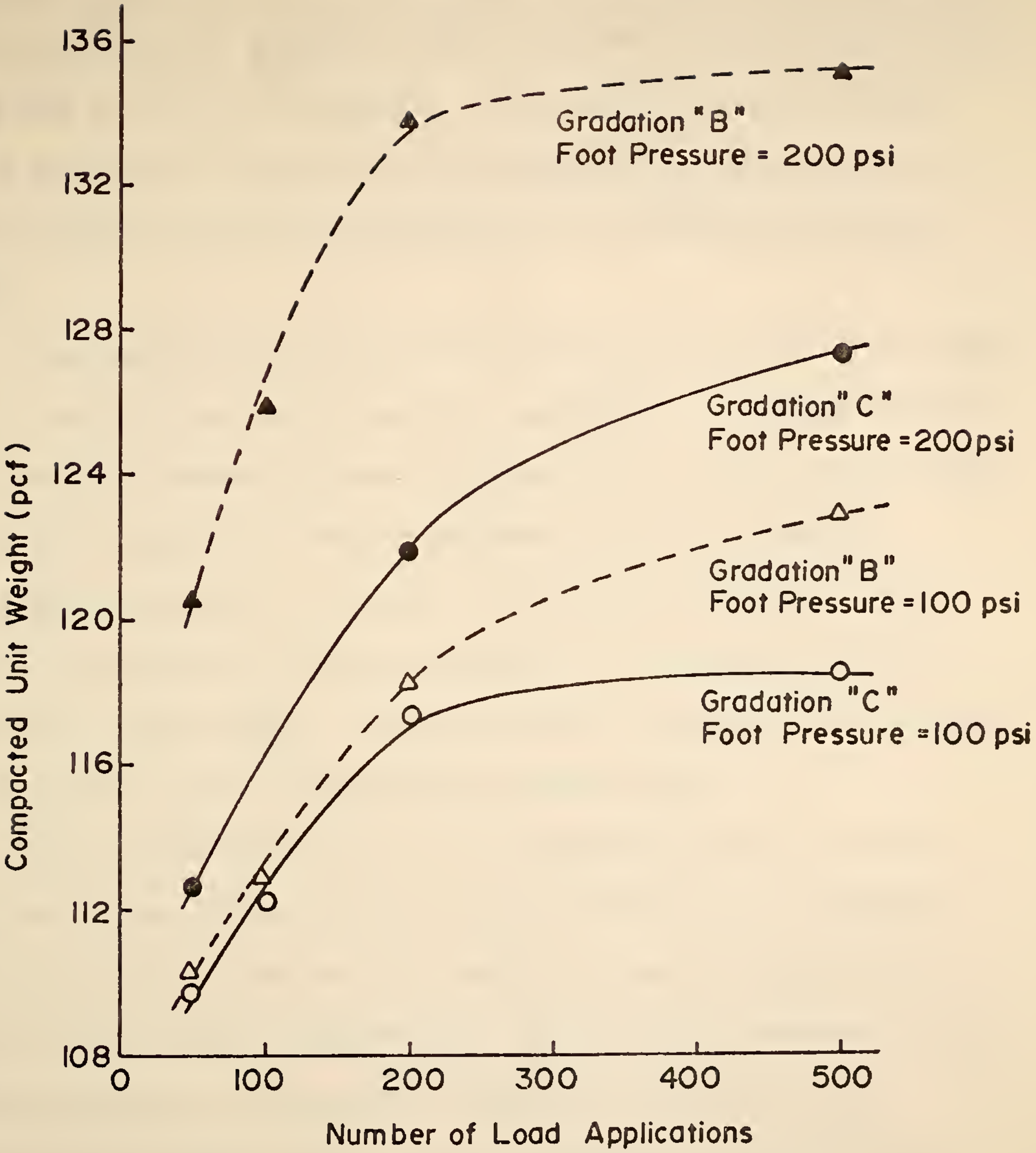


FIGURE 15 EFFECT OF INITIAL SAMPLE GRADATION AND KNEADING COMPACTION FOOT PRESSURE ON AVERAGE COMPACTED UNIT WEIGHT.

evident in the tests performed using a foot pressure of 50 psi however. In all cases, the "A" gradation samples resulted in higher unit weights than either the "B" or "C" samples. This is due to the availability of fine particles to fill the larger interparticle voids, and will be discussed further following presentation of the degradation analysis results.

In the compaction results described above, the effect of the shale sample's moisture content has not been considered. A primary function of the kneading compaction tests performed in this investigation was to measure and evaluate the particle degradation due to compaction. Separation of particles by sieving was found to be impossible after water had been mixed into the shale samples in the manner used for conventional, Proctor type, compaction tests. Thus compaction moisture content was not varied independently in these tests.

For the kneading compaction tests reported herein, the shale particles had an average moisture content of 4.3%, with a standard deviation of ±1.2%. This range in moisture content represents the variation in the stored shale samples, and is not the result of intentional manipulation. The effect of moisture on the compaction properties of the Attica model shale is discussed in Sections 4.1.3 and 4.4.3. For the relations described above, the effect of sample moisture content appears small and is inconsistent over the range of tests for which it was measured.

Figure 16 shows the trend in particle degradation, as indicated by average values of the index of crushing, for samples tested with a compaction foot pressure of 100 psi. Excepting various anomalous

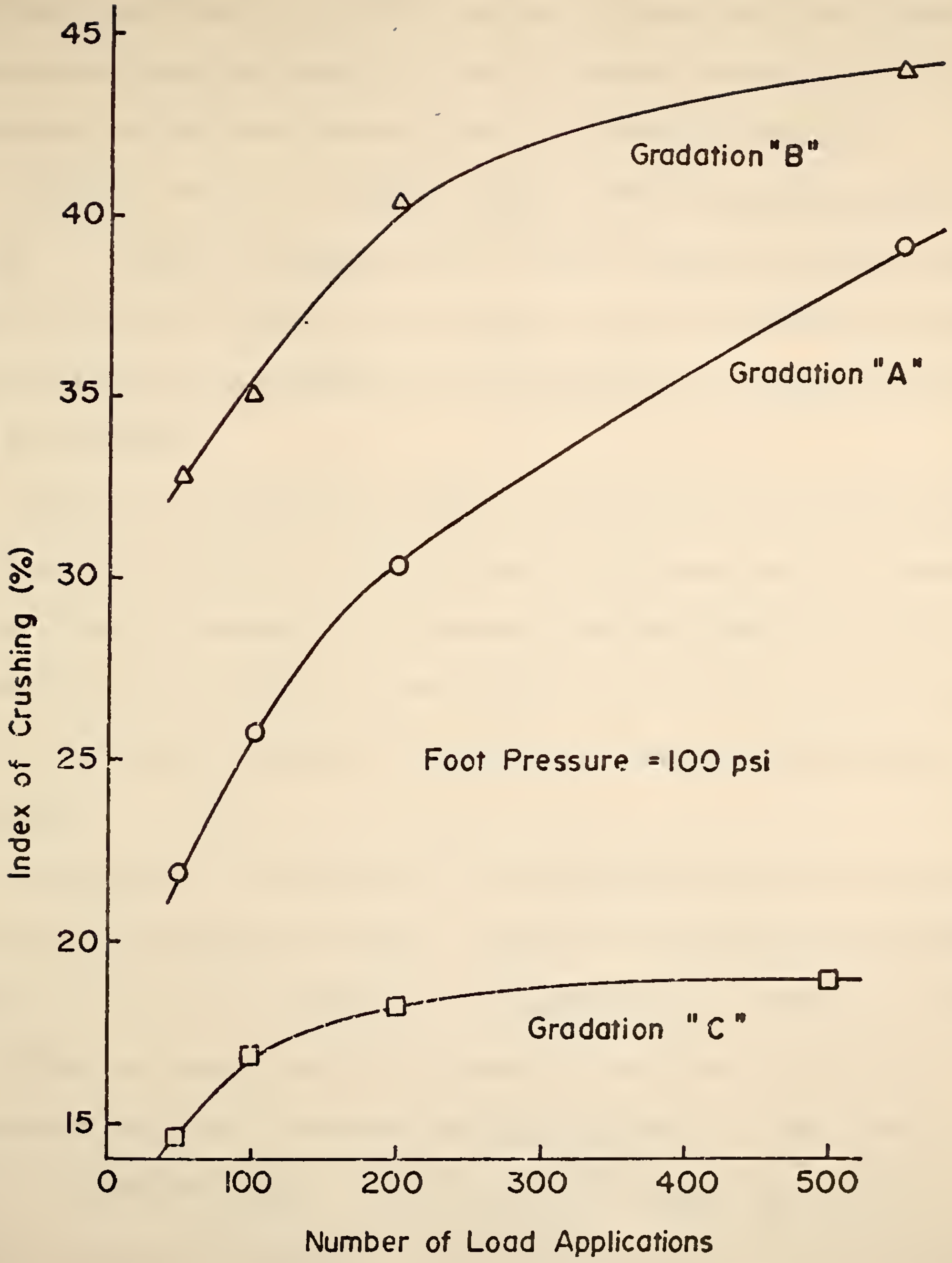


FIGURE 16 EFFECT OF INITIAL SAMPLE GRADATION ON PARTICLE DEGRADATION FOR KNEADING COMPACTION.

test results, the trends shown in Figure 16 are the same for all three gradations regardless of whether the index of crushing or the change in aggregate gradation modulus is used to measure degradation. The IC data appear to show more scatter than do the AGM data however.

Figure 17 shows the average change in AGM for samples with the "B" and "C" initial gradations over a range of compaction effort levels. From this family of curves it can be seen that particle degradation is influenced more by the compaction foot pressure than by the number of load applications.

Results of degradation analyses using both the IC and AGM methods show that the "B" initial gradation samples underwent more degradation than did the "C" samples. This suggests that the higher unit weights measured for the "B" tests are the result of a greater relative production of fine particles which filled the larger interparticle voids during compaction.

An analysis or discussion of the mechanics of particle breakage is beyond the scope of this work. However the difference in degradation between the "B" and "C" samples at the same levels of compactive effort may indicate an upper bound to the specific magnitude of particle degradation that can be achieved through application of any specific level of compaction effort. This upper limit to particle degradation is also evident for increases in the compaction effort. The degradation curves shown in Figure 17 all appear to flatten, indicating a decrease in the rate of degradation, as the number of load applications becomes large. The same tendency is somewhat evident in Figure 18 for increases in compaction foot pressure.

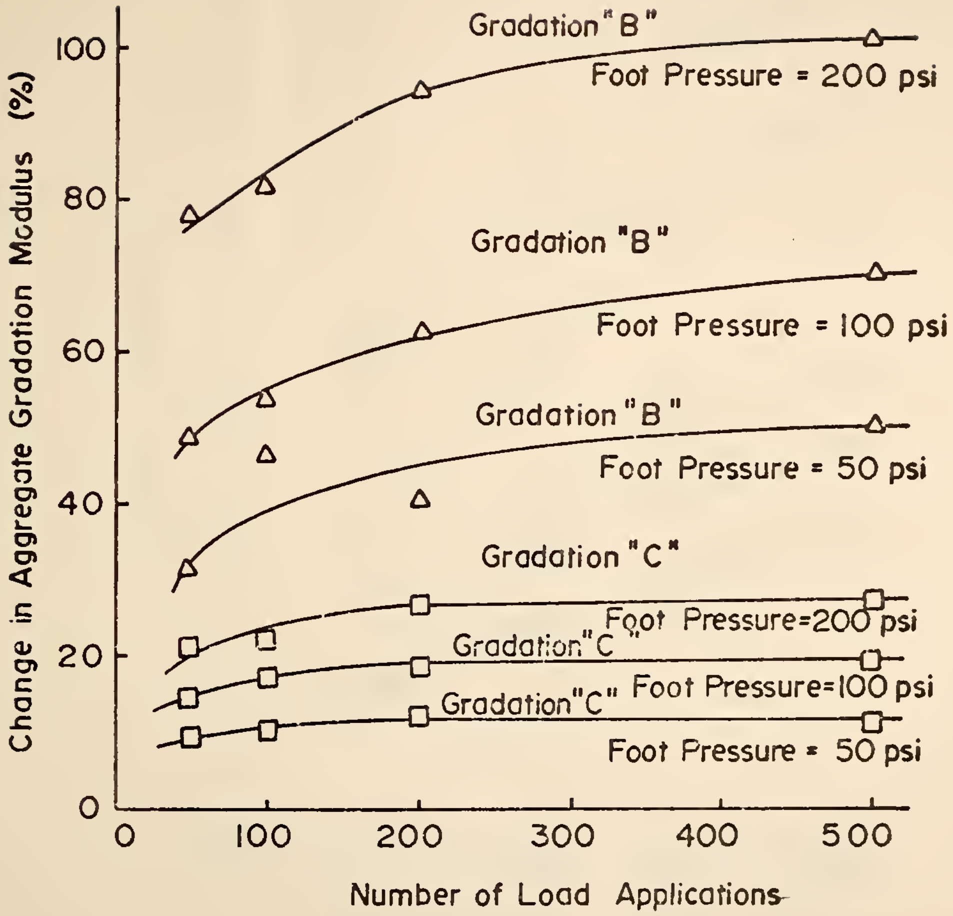


FIGURE 17 EFFECT OF INITIAL PARTICLE SIZE AND KNEADING COMPACTION EFFORT LEVEL ON AVERAGE PARTICLE DEGRADATION.

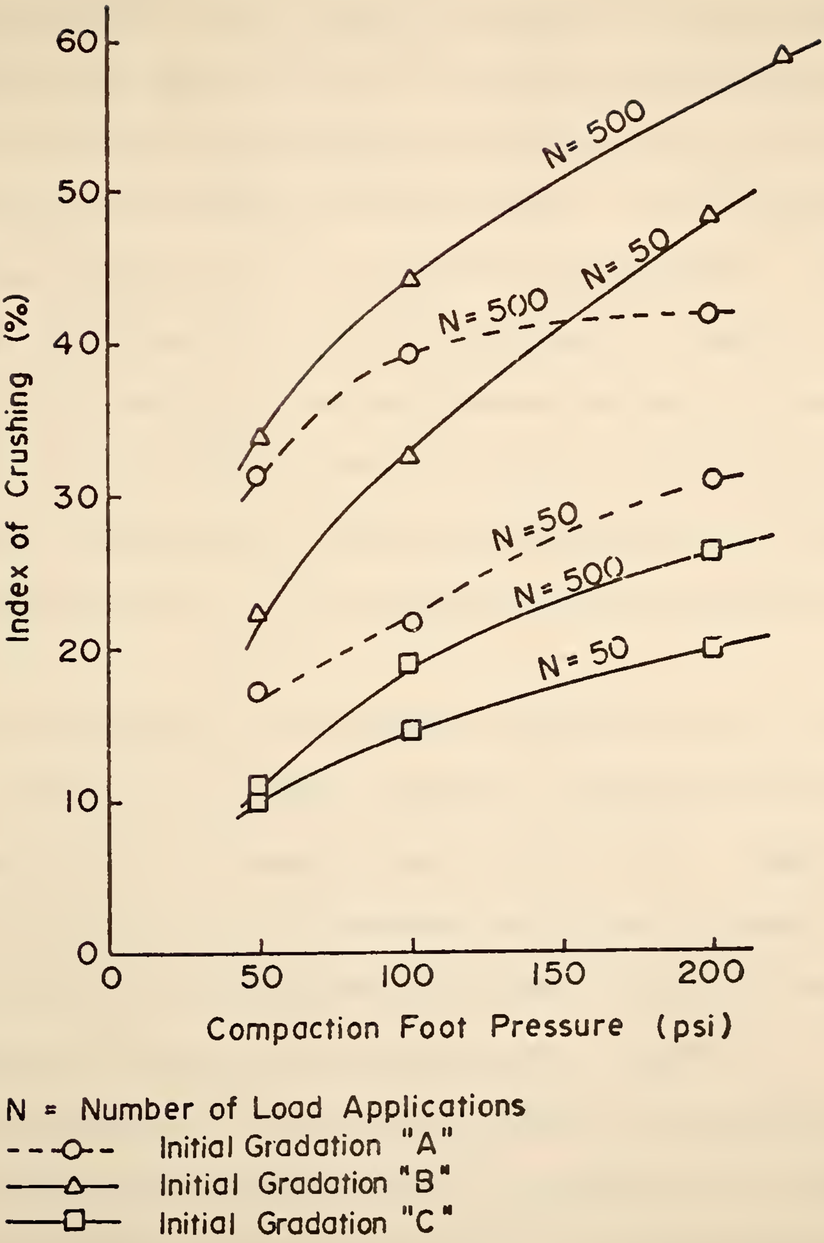


FIGURE 18 EFFECT OF INITIAL PARTICLE GRADATION AND KNEADING COMPACTION EFFORT LEVEL ON AVERAGE PARTICLE DEGRADATION.

The "B" gradation samples show more degradation than do the "A" samples, as well as a higher rate of gradation change with increases in compaction effort. The "A" gradation samples resulted in higher compacted densities than the "B" and "C" samples, probably because of the initial presence of a substantial percentage of fines. Observations during the kneading compaction tests showed that as sample density increased, the penetration of the compaction foot decreased. This suggests that higher density restricts the movement of particles within the sample during compaction, and thus reduces the particle degradation due to abrasion or "kneading". From this it can be hypothesized that a real limit to shale particle degradation exsists under a specific level of compaction effort, and is dependent upon the density of the sample.

Statistical analyses of the kneading compaction results were peformed to determine the specific effect of compaction effort on both the densification and degradation of Shale No. 11. Computerized regression analysis was used to determine the degree of linear correlation between the independent and dependent variables, and to generate equations describing this correlation. Pearson's correlation coefficient, "R", was used to measure correlation following the method described by Draper and Smith (25) and Nie et al. (60).

Regression analysis was performed on the "A" gradation test results. This data group consists of the results of 25 tests, however analyses which include the effect of shale moisture content are available for only 15 tests. Results described below indicate that the effect of moisture was not significant over the range tested, and results of

analyses for a group population of 15 samples approximate the results from the larger group.

Initially, regressions were performed to define the relation between single pairs of independent and dependent variables only. These variables are defined in Table 5.

Using the "t" statistic described by Nash (59), correlations were considered significant when the "R" correlation coefficient exceeded a minimum value based on the 99% probability level. For a comparison of any two variables, this resulted in a minimum value of $|R| > 0.51$ for 25 samples, and $|R| > 0.64$ for the smaller group of 15 samples. An absolute value of R equal to 1.00 indicates perfect linear correlation (59). The correlation coefficients resulting from comparison of the primary kneading compaction variables are shown in Table 6, based on a population of 25 samples.

The following conclusions can be made on the basis of the $|R|$ values shown in Table 6.

1. All of the dependent variables (DM, IC, AA, PA) are more closely related to the compaction foot pressure, P, than to the number of load applications, N, as indicated by higher values of $|R|$. This agrees with the qualitative observations made previously.

2. Of the three degradation terms, the index of crushing shows the highest correlation with P, but the lowest with N. The arithmetic change in the aggregate gradation modulus shows the highest correlation with N. None of the comparisons indicate the PA term is particularly useful in describing the degradation resulting from compaction, compared to the IC or AA terms.

Table 5.

Variables Used In Kneading Compaction Analyses.

Primary or Measured Variables:

Symbol	Description	Units
P	Compaction Foot Pressure	psi
N	Number of Load Applications	
w	Moisture Content	%
WT	Sample Weight	grams
DM	Compacted Moist Unit Weight	pcf
DD	Compacted Dry Unit Weight	pcf
IC	Index of Crushing	%
AA	Arithmetic Change in AGM	
PA	Percent Change in AGM	%

Secondary or Computed Variables:

$X1 = P \times N$

$X2 = (P \times N) \div DM$

$X3 = (P \times N) \div DD$

$X4 = P \div DM$

$X5 = P \div DD$

$X6 = N \div DM$

$X7 = N \div DD$

$X8 = (P \times N) \div WT$

LOG_{10} of: P, N, DM, DD, IC, AA, PA, X1, X2, X3, X8

LOG_{e} of: P, N, DM, DD, IC, AA, PA, X1, X2, X3, X8

SQUARE ROOT of: N, P, DM, DD, IC, AA, PA, X1, X2, X3, X8

Table 6.

Absolute Values of Significant Correlation Coefficients For Pairs of Kneading Compaction Variables.

N	-					
WT	-	-				
DM	0.74	0.54	*			
IC	0.68	0.57	*	0.93		
AA	0.66	0.61	*	0.91	0.99	
PA	0.63	0.60	*	0.87	0.97	0.99
	P	N	WT	DM	IC	AA

- Indicates No Rational Basis For Correlation

* Indicates No Significant Correlation, ie. $|R| \leq 0.51$

3. The weight or size of the sample does not appear related to either the compacted unit weight or the particle degradation, within the range tested. Sample weights were only slightly varied however. For these 25 tests, the mean sample weight was 881 grams, with a standard deviation of ±60 grams. Multiple variable regression analyses, described below, also resulted in a low significance for the effect of sample weight.

4. All three of the degradation terms showed a high degree of correlation, $|R| \geq 0.87$, with the compacted unit weight.

5. The three degradation terms all appear to be closely interrelated. The strong correlations, $|R| \geq 0.97$, indicate that an analysis based on any one of the three terms would be approximately correct for either of the other degradation terms also.

Other correlations using the logarithm or square root of the primary variables did not yield higher "R" values than those shown in Table 6.

In order to determine the effect of sample moisture content, correlation coefficients were also calculated for pairs of primary variables as shown in Table 7. Sample moisture content was only measured for 60% of the tests previously described. Initial kneading compaction tests were performed with the erroneous assumption that the shale moisture content had stabilized in the storage bin and could be considered a constant factor. This problem is discussed further in Appendix A. The results shown in Table 7 are based on a statistical population of 15 samples.

Table 7 shows that when treated as an independent variable, the

Table 7.

Absolute Values of Significant Correlation Coefficients For Pairs of Kneading Compaction Variables Including The Effect of Sample Moisture.

P	-	*	*
N	-	0.65	*
WT	-	*	*
IC	*	0.92	0.91
AA	*	0.92	0.90
PA	*	0.85	0.83
DM	*	-	-
DD	*	-	-
	W	DD	DM

- Indicates No Rational Basis For Correlation

* Indicates No Significant Correlation, ie. $|R| \leq 0.64$

sample moisture content was not significantly related to either the compacted unit weight (DM or DD) or to any of the three particle degradation terms. This apparent anomaly may be explained by considering that the sample moisture varied only slightly (mean value equal to 5.2%, standard deviation equal to ± 1.2%, for the "A" samples).

Table 7 also shows that the degradation results were more closely related to the dry unit weight, DD, than to the moist unit weight, DM, for the same sample group. The difference in "R" values is small however and may not be significant. A statistical group of more than 15 samples, and a larger variation in moisture content would be required to further define the statistical effect of sample moisture in kneading compaction tests.

The kneading compaction test results for gradation "A" were also subjected to multiple variable regression analyses to determine equations relating the observed data. These analyses were divided into groups of regressions intended to to describe physical models of the compaction process. The second power of the correlation coefficient, R^2, is used to indicate the degree of compatability of the data with the equations determined. R^2 values are more sensitive to small differences than are the root values (59). Analyses were performed using both the primary and secondary variables, as defined in Table 5.

The first model to be tested was intended to relate the compaction effort with the resulting sample unit weight. For a population of 25 samples, the best derived equation was:

$$DM = 106.9 + 0.093(P) + 0.024(N) \qquad R^2 = 0.83$$

For the smaller group, the dry unit weight was considered as the dependent variable and the following equation was developed.

$$DD = 102.4 + 0.074(P) + 0.025(N) \qquad R^2 = 0.82$$

A second regression model considered the use of the sample weight to normalize the compaction effort. The best equation for this model is:

$$DD = 2.95 \left[\frac{P \times N}{WT}\right]^{1/2} + 102.0 \qquad R^2 = 0.81$$

The correlation for this type of model was slightly improved when the weight was entered as an independent term, resulting in the following equation.

$$DM = 122.8 + 0.090(P) + 0.023(N) - 0.017(WT) \qquad R^2 = 0.85$$

When entered in this manner, the sample weight improved the overall correlation slightly, but when considered uniquely, WT was the least significant of the three independent variables.

In the same way, the shale moisture content, w, was considered both as an independent variable, and as a component in calculating the dry unit weight term. For these two conditions, the best correlation resulted from treating w independently, as follows.

$$DM = 161.3 + 0.075(P) + 0.72(w) + 0.024(N) - 0.064(WT) \qquad R^2 = 0.86$$

Two models were tested to describe the particle degradation that occurred during kneading compaction. The first of these, a compaction effort degradation model, resulted in the following relation.

$$AA = 0.312 + 0.0023(P) + 0.00077(N) \qquad R^2 = 0.80$$

Comparable equations were obtained using the IC or PA terms instead of the arithmetic change in AGM, but the correlations were somewhat less. The other model attempted to relate compacted density with

degradation, with the following results.

$$IC = 1.17(DM) - 113.5 \qquad R^2 = 0.86$$

$$IC = 1.20(DD) - 109.7 \qquad R^2 = 0.84$$

Finally, an "all-terms" analysis was performed to relate the compaction effort, sample weight and moisture content, with the compacted unit weight and degradation. In making such an analysis, the unit weight and degradation were considered interchangeably as the dependent variable. The best result was:

$$IC = -50.3 + 0.023(P) + 0.0091(N) - 0.015(WT) + 0.72(DM) \qquad R^2 = 0.88$$

for the larger sample population, and:

$$\begin{aligned} AA = 0.561 &- 0.00045(P) - 0.00022(N) + 0.0076(w) \\ &+ 0.00090(WT) - 0.018(DD) \qquad R^2 = 0.86 \end{aligned}$$

for the smaller population. For the latter equation, the sample weight and moisture content were found to be the least significant variables.

Two factors must be considered in evaluating the equations and correlations described above. First of all, the regressions were performed using a small number of tests (25 or 15) to describe the effects of a relatively large number of variables, i.e. two compaction effort terms, compacted unit weight, and particle degradation. Furthermore, the shale specimen weight and moisture content were allowed to vary in an uncontrolled manner. This is not consistent with good statistical experiment design (60).

Even with these qualifications, the analyses described above are of value. The regressions presented in this section may be used to plan further experimental work with the kneading compactor.

Furthermore, it should be evident that results of even a limited

number of tests can be used to generate relations describing the compaction of a shale material. The object of any field test pad should be to generate information that can be used in subsequent embankment design. Relations such as the ones presented herein are indicative of the analyses that could be performed on shale test pad data.

4.1.4 Conclusions

At this time no direct comparisons can be made between the results of kneading compaction tests and field compaction of shales. Such a correlation may be possible when further field compaction data are available.

From the discussion above, three conclusions can be drawn.

1. It appears that sample densification and degradation are interrelated, and dependent upon both the magnitude of compaction effort and the initial particle gradation.

2. The densification and degradation of shale samples appear to be self-limiting reactions, regardless of compaction effort magnitude or initial gradation. Non-linear regression analyses could not be performed however, so this trend was not quantified.

3. The analyses which were presented could be repeated using field compaction data from shale test pads. The use of such analyses, based on real compaction conditions, could enable selection of compaction methods and equipment to achieve rational design expectations for shale embankment construction.

4.2 Gyratory Compaction

4.2.1 Introduction

The gyratory compaction testing machine was developed by the Corps of Engineers to provide a testing and design procedure for hot-mix bituminous paving mixtures. The concept of the apparatus follows a compaction method originally used by the Texas Highway Department (13). In addition to assessing the compaction characteristics of bituminous mixtures, the apparatus has been used for compaction, repetitive loading, and shear testing of soils, crushed rock and aggregate mixtures (56, 58, 78). The various aspects and uses of the gyratory compaction apparatus have been described by McRae and others (55, 72).

The use of gyratory compaction tests to investigate the degradation of shales used in highway construction was pioneered by the Bureau of Materials, Testing and Research, of the Pennsylvania Department of Transportation (69, 70, 78). That work developed a test to distinguish shales which would rapidly disintegrate through weathering action, involving both slaking in water and gyratory compaction, to evaluate relative durability. There is no record of tests intended to simulate the results of field compaction of shales.

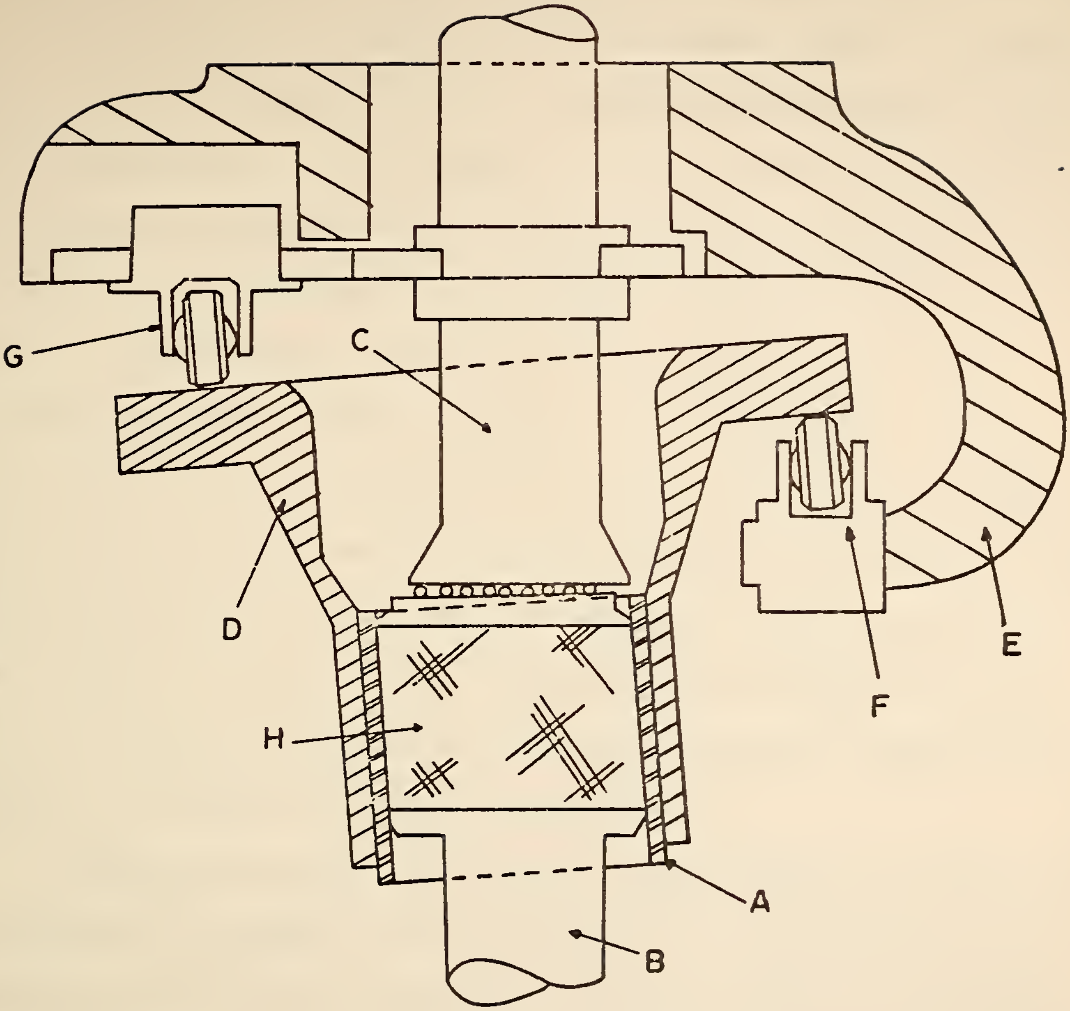
Current use of gyratory compaction tests for paving mixtures tends to emphasize the relation of the tests to actual field behavior (4, 55). Gyratory compaction causes particle movements in test specimens which closely resemble the effect of repetitive traffic loading on highway pavements (55). The purpose of the gyratory testing of shales reported

herein was to measure compacted densities and particle degradation in a controlled manner, and to determine whether the test-induced densification and degradation can be considered analogous to the effects of field compaction. Tests were conducted using the "Attica" model shale, Shale No. 11.

4.2.2 Description of the Apparatus and Test Procedure

Figure 19 shows a diagrammatic section through the gyrating mechanism of the gyratory compaction machine. The compaction sample is placed in a loose condition in the open-ended cylindrical mold which is then fitted into the mold chuck. The lower axial ram is raised hydrauically to confine the sample against the upper axial ram, and to secure the mold while the cover of the mold chuck is bolted. During this process some care is needed to prevent the loss of fines from the mold, and to avoid particle segragation within the mold. Since the sample is inaccessible to observation after placement in the mold, particle segragation can only be evaluated after compaction when the sample is removed from the mold. Despite special care in preparation and loading the samples, a high rejection rate resulted from observations of noticeable particle segragation. The loss of fines was minimized by surrounding the sample with a plastic sample bag and inserting a cardboard disk at each end. The cardboard disks were replaced after each test, so that whatever influence the disks exerted on the degradation process may be assumed constant for all of the tests.

Some densification and degradation of the shale resulted when the sample was confined by the pressure of the lower axial ram before



A. Sample Mold

B. Lower Axial Ram

C. Upper Axial Ram

D. Mold Chuck

E. Rotating Arm

F. Lower (Fixed) Roller

G. Upper (Pressure) Roller

H. Compaction Sample

FIGURE 19 DIAGRAMMATIC SECTION THROUGH GYRATING MECHANISM OF GYRATORY COMPACTION APPARATUS. AFTER MOAVENZADEH (58).

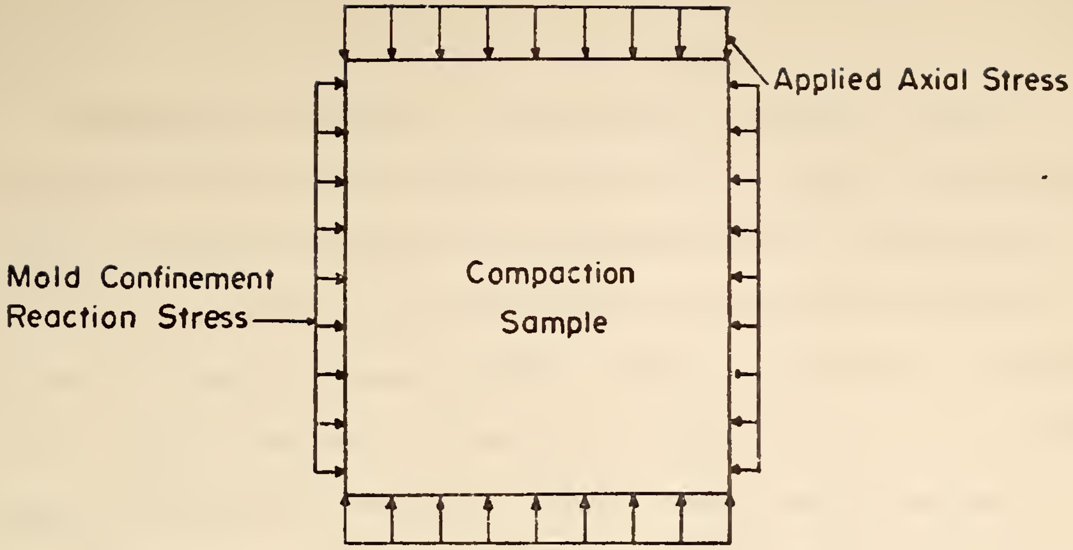
the gyration was started. This is discussed in Section 4.3.

After pressure is applied axially the gyrating process is begun when an electric motor is turned on, causing the rotating arm apparatus to move around the confined specimen. The two rollers (one at either end of the rotating arm) force the mold chuck to tilt along the axis of the arm. The eccentricity (amount of tilt) of the mold chuck is controlled by the relative vertical position of the rollers, and is termed the initial angle of gyration. For the apparatus used in this study, the angle of gyration had a range of zero to three degrees above the horizontal plane, but was set at one degree for all the tests reported herein. The initial angle of gyration was set for an axial ram pressure of 50 psi. Roller bearings between the upper axial ram and the loading platen insure that the axial pressure is continuously applied in a vertical direction during the test.

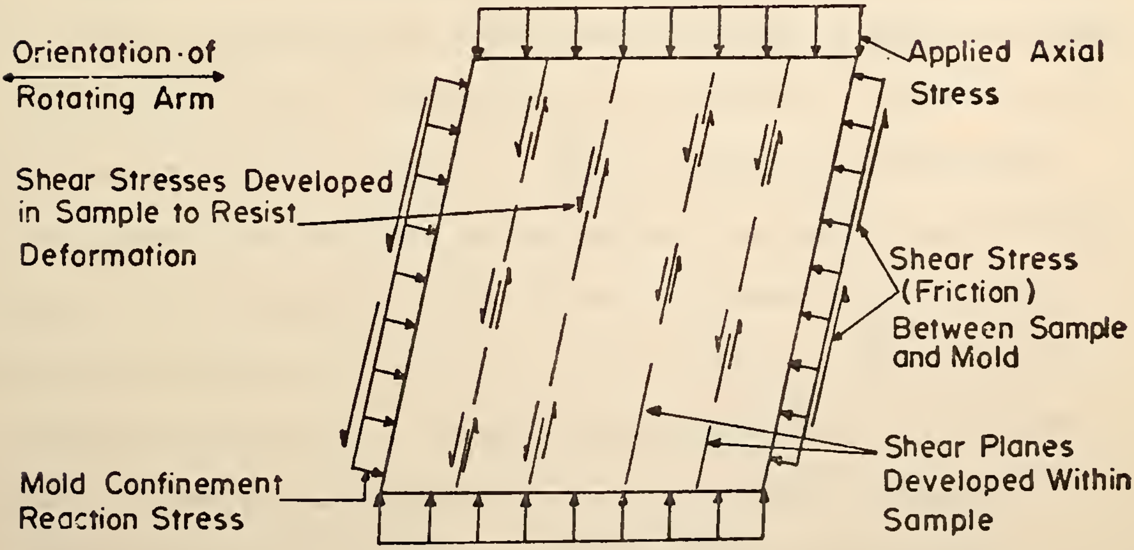
Although the rollers provide only a one dimensional constraint on the movement of the mold chuck, the rotation of the arm continuously changes the horizontal orientation of this constraint. The mold and sample tilt with respect to the vertical ram axis, but do not otherwise move. The rotating arm and mold chuck are dimensioned so that the compaction sample is continually tilted about its own center of gravity in a constantly changing horizontal direction. The rollers allow complete freedom of response in rotational movement perpendicular to the axis of the rollers (or in general, perpendicular to the rotating arm) within the limits imposed by the test frame, i.e. about $\pm 5^\circ$ The extreme limits of this three-dimensional tilting are resisted by shearing stresses developed within the sample. These shearing stresses

result from the sample's resistance to the deformation induced by the tilting of the mold and sample with respect to the applied vertical load. Figure 20 shows this stress condition as described by McRae (55). The shear stress developed within the sample may vary considerably during the test. The maximum shear stress can be compared with shear strain or the magnitude of vertical ram load ("normal stress"), but such a comparison does not appear directly relatable to the concept of shear strength as it is used in conventional soil mechanics.

The shear stress developed within the sample can be indirectly measured by the fluid pressure developed in an oil or air filled envelope in the upper roller. The fluid pressure in the upper roller increases as the vertical load on the roller increases, indicating greater resistance to deformation within the sample. The magnitude of the fluid pressure and the measured deflection (tilt) of the mold chuck can be used to develop a moment equation which can be solved for the shear resistance of the sample. McRae's solution for this equation (55) includes a term for the friction between the mold and sample, however work by Ruth and Schaub (72, 73) indicates that this friction can generally be ignored. It is significant to note that McRae's solution depends on the static friction between mold and sample, and no actual measurements of the dynamic or gyratory friction have been reported.



(a.) Stress Condition for Confined Sample Before Gyration.


Orientation of Rotating Arm
Applied Axial Stress
Shear Stresses Developed in Sample to Resist Deformation
Shear Stress (Friction) Between Sample and Mold
Shear Planes Developed Within Sample
Mold Confinement Reaction Stress

(b.) Instantaneous Stress Condition During Gyration.

FIGURE 20 FREE BODY DIAGRAM OF GYRATORY COMPACTION SAMPLE BEFORE AND DURING GYRATION. AFTER McRAE (55).

4.2.3 Gyratory Shear Tests

Regardless of the method of calculation, the sample's shear resistance during gyration appears dependent on the axial load, number of roller arm revolutions, and test specimen material. Although not yet isolated and tested, the material characteristics which control the gyratory shear resistance probably include the effects of particle size and shape, gradation, and compacted density, as well as the particle strength and roughness, as in conventional shear strength testing. Gyratory shear tests were performed following the method described by McRae (55).

Figures 21 and 22 show two plots of applied vertical stress and the resulting gyratory shear resistance, calculated by using the methods of McRae (55), and Ruth and Schaub (72), respectively. Data for both plots come from the same series of tests on the Attica model shale. The term for static friction between the mold and sample has not been included in the calculations for Figure 21, because of the cost and difficulty of measuring this value. If included, the friction term would tend to reduce the calculated shear resistance, i.e. bring the results of both calculation methods closer together. McRae has suggested that such plots can be interpreted as a gyratory shear version of the usual Mohr-Coulomb shear failure envelope used in soil mechanics (55). If so used, the angle of gyratory shear friction, ϕ_G, for the model shale is seen to vary from 29° to 19° (McRae's method) and 14.5° to 10.5° (method of Ruth and Schaub).

The validity of such an interpretation can be questioned.

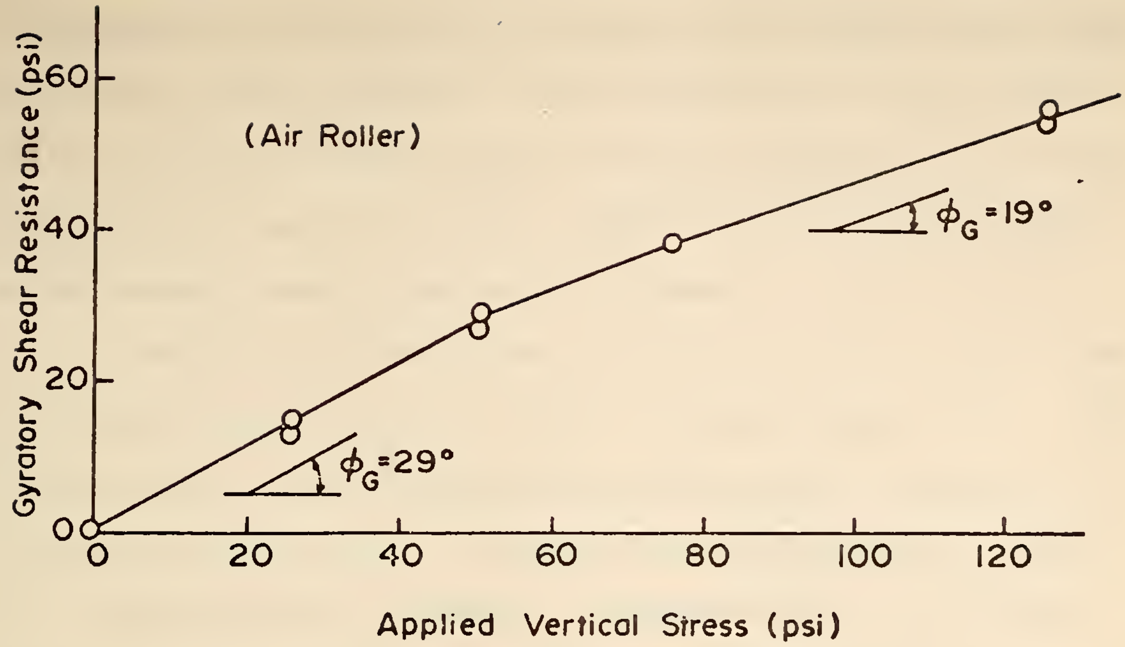


FIGURE 21 GYRATORY SHEAR RESISTANCE CALCULATED BY METHOD OF McRAE (55) FOR ATTICA SHALE.

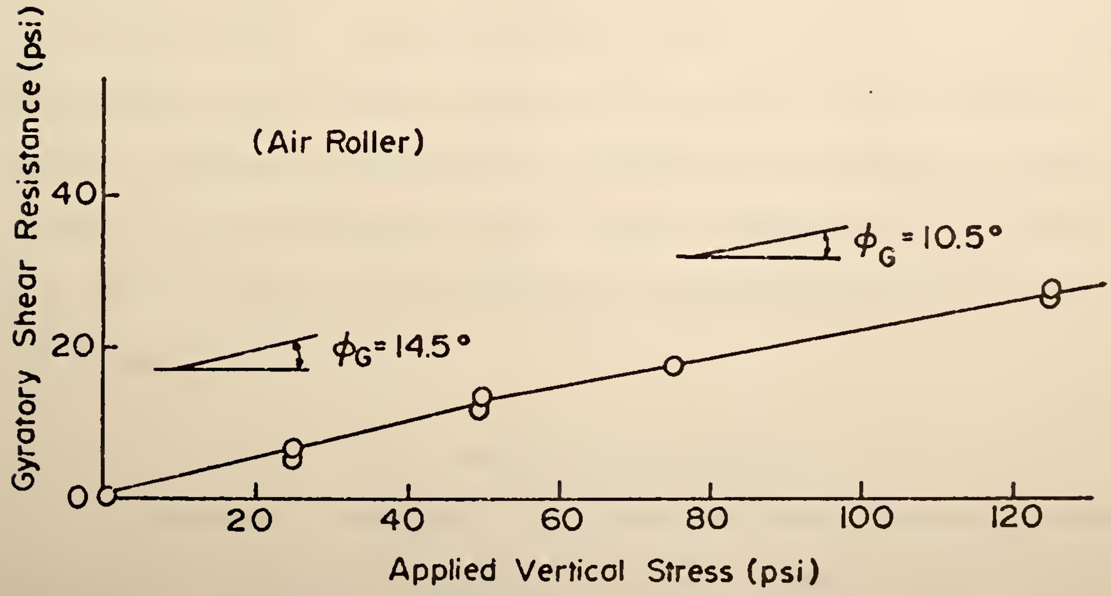


FIGURE 22 GYRATORY SHEAR RESISTANCE CALCULATED BY METHOD OF RUTH AND SCHAUB (72) FOR ATTICA SHALE.

A similar series of tests performed on commercially available Ottawa quartz sand resulted in ϕ_G values of 24.5° (McRae's method) and 16° (method of Ruth and Schaub). These values are both well below the expected values from direct shear or triaxial shear tests for the same material at the same density (52). Further measurements and analysis of the gyratory shear resistance are discussed in the next section.

As mentioned previously, the resistance to gyratory shear can be indirectly measured by a fluid pressure cell in the upper roller. This cell may contain either air or hydraulic oil, or may be replaced by a fixed position roller which will not allow the upper roller to "back up" as shear resistance increases. According to McRae (55, 56) the fixed roller is usually used for kneading type compaction tests on soils where compacted density is the primary variable of interest. Both the oil and air pressure cell rollers enable the degree of kneading or deformation to vary and to be measured during the test. If the resistance to kneading or densification increases sufficiently, the pressure cell roller will back up to the point where compaction ceases. This can be considered as comparable to a sheepsfoot roller "walking out" of a compacted lift. Other investigators have noted that the air roller is more sensitive to stress changes than is the oil roller (55).

4.2.4 Compaction Test Results

In addition to the shear tests described above, gyratory compaction tests were performed to determine the relation between compaction effort, gyratory shear resistance, compacted unit weight and sample degradation.

The compaction effort was varied by changing both the ram pressure and the number of gyratory revolutions. Tests were performed on the Attica shale, gradation "A", using both the air and oil rollers. Following the method of Ruth and Schaub (72), the gyratory shear resistance was calculated as a ratio of the roller pressure and incremental sample height, multiplied by a constant.

Table 8 shows the compaction effort matrices which describe the gyratory compaction test program. Test results are tabulated in Appendix D. Except for four tests using the air roller, all of the gyratory compaction samples were analyzed for particle degradation. The four air roller tests which were continued for 500 revolutions were performed only to measure and compare the effects of densification and gyratory shear resistance as a function of the number of gyratory revolutions.

Gyratory compaction with both the air and oil rollers produced about the same magnitudes of compacted dry unit weight at comparable effort levels. This is shown in Figure 23. Although both rollers produced about the same average unit weight values, the data from the oil roller tests seem to show more scatter. Figure 23 shows that both rollers densify the shale samples at a decreasing rate for increasing magnitudes of ram pressure.

Figure 24 shows the effect of increasing the number of gyratory revolutions on both the compacted unit weight and gyratory shear resistance of four samples compacted using the air roller and a ram pressure of 50 psi. Although there are significant differences in the compacted unit weight curves for the four samples, all show about the

Table 8.

Gyratory Compaction Test Program,
Compaction Effort Matrices.

Oil Roller

Ram Pressure (psi)	Number of Revolutions			
	10	25	50	100
25			2	2
50	3	2	3	3
75			3	4
100			3	3

Air Roller

Ram Pressure (psi)	Number of Revolutions				
	10	25	50	100	500
25			2		
50	2	2	2	2	4
75			2		

2 Indicates Number of Tests Performed

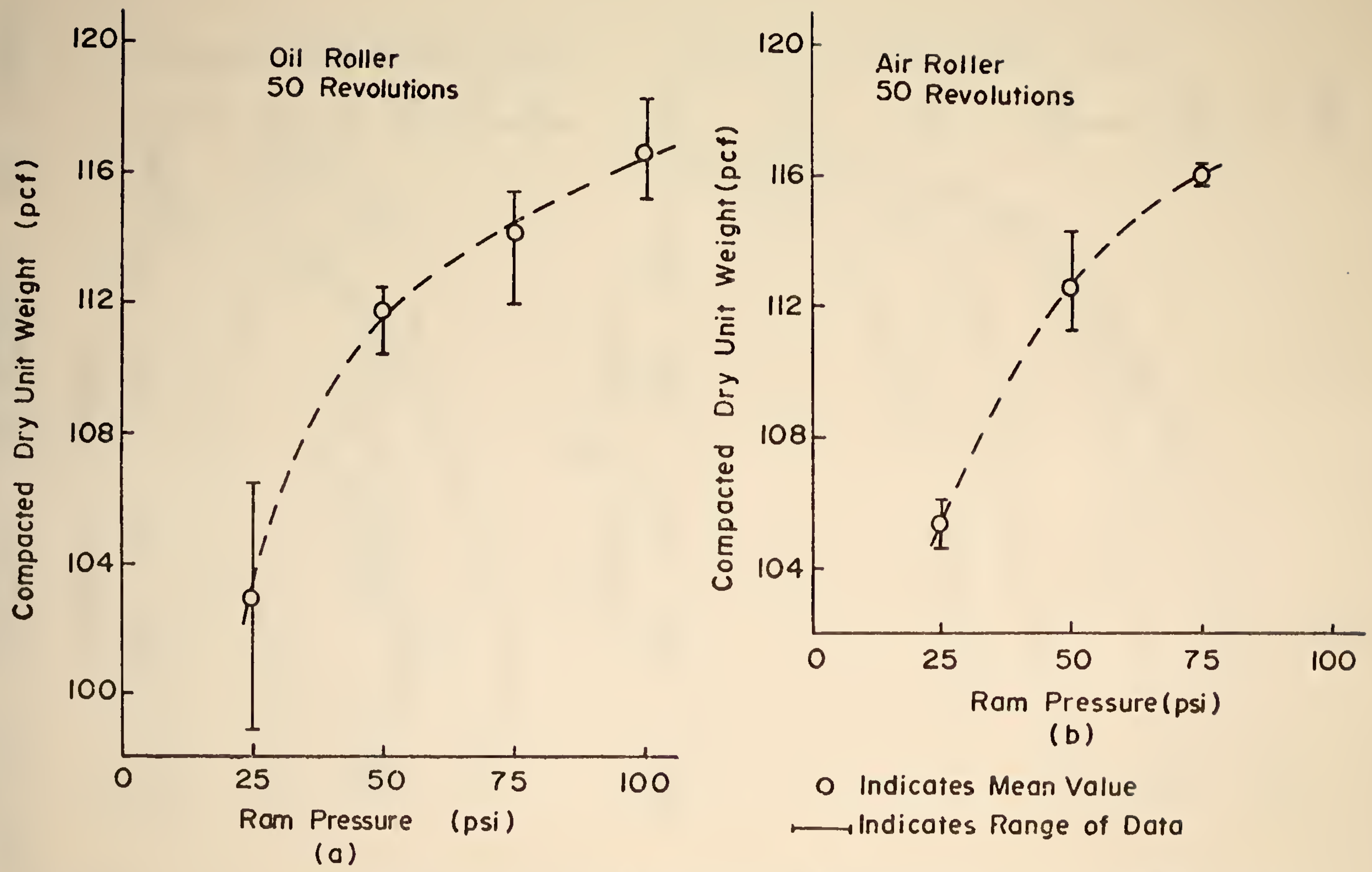
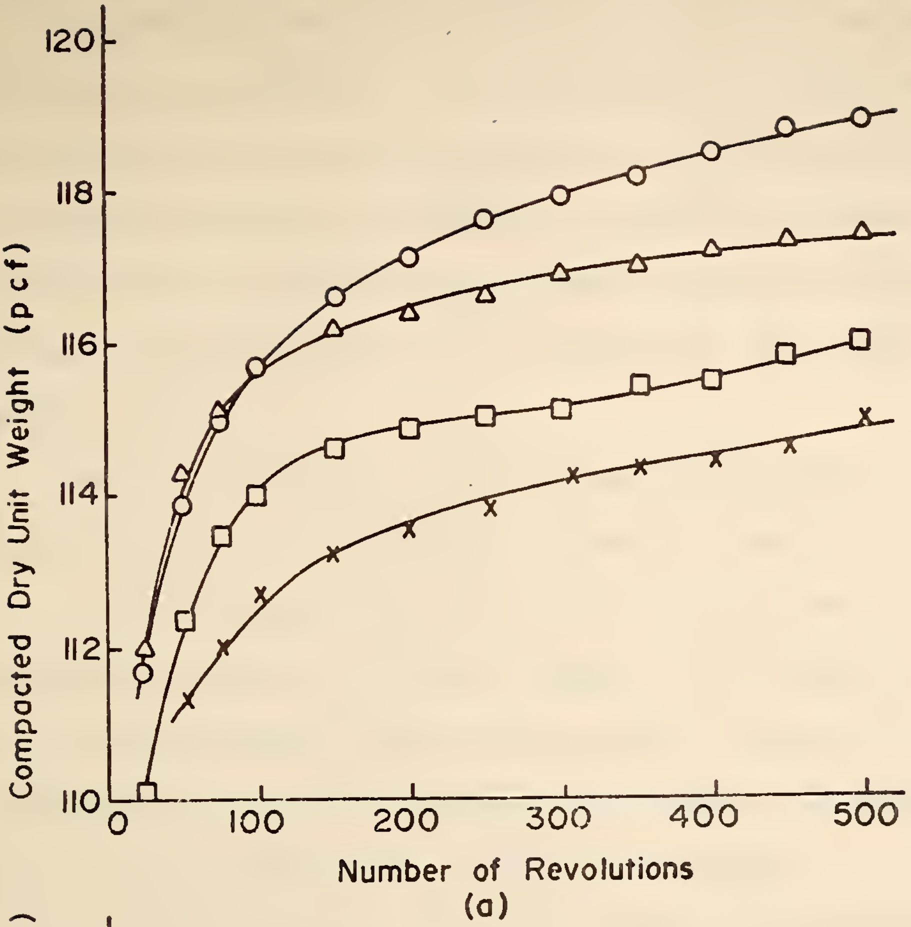
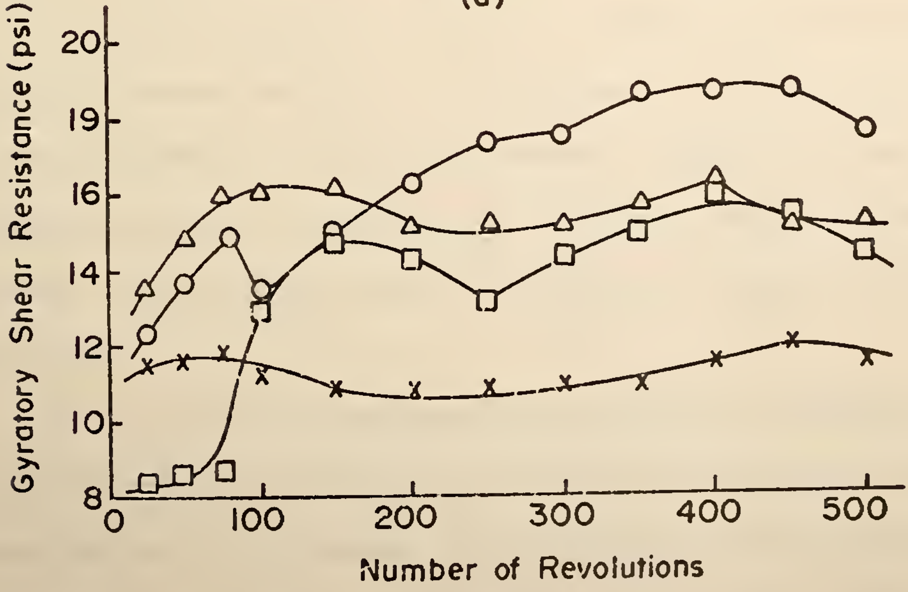


FIGURE 23 COMPARISON OF COMPACTED UNIT WEIGHTS OBTAINED WITH GYRATORY COMPACTION AIR AND OIL ROLLERS.



(a)



(b)

FIGURE 24 COMPACTED UNIT WEIGHT AND GYRATORY SHEAR RESULTS FROM GYRATORY COMPACTION USING THE AIR ROLLER.

same trend towards a decreasing rate of densification.

In contrast, there is very little similarity in the gyratory shear resistance developed in the four samples. The data presented in Figure 24b show no simple linear change in gyratory shear resistance as the compaction effort is increased. Nonlinear stress-strain response is characteristic behavior for all granular materials subjected to confined compression, and has been described as "stick-slip" behavior (52).

The shear resistance cycle developed during gyratory compaction can be thought of in three distinct stages, as described below.

1. Initial application of the axial ram pressure acts to confine the sample in the mold. This confinement under pressure forces the shale particles to move into an array capable of resisting further deformation due to the ram pressure. In essence, the particles are "locked" into an arrangement to resist deformation.

2. When the gyratory process is started the confined sample is forced to deform, leading to the creation of internal shear planes as shown in Figure 20. With the development of shear stresses within the sample, the forces acting upon the individual shale particles are changed. The changing forces act to deform, rearrange and degrade the particles.

3. As gyration continues these processes (deformation, rearrangement and degradation of the particles) result in progressive densification of the sample. The resistance to these processes is measured by the fluid pressure developed in the upper roller.

The variation in gyratory shear stress during densification may

be due to a combination of two basic reactions. During the gyration process the sample is repeatedly subjected to shearing stresses along an infinite number of planes (55). The gyration process can be considered as a cyclic loading phenomena. Each gyrating arm revolution forces the sample to deform in a continuously repeated sequence due to the transient load applied around the sample's circumference. During this sequence the contacts between adjacent shale particles are subjected to both elastic and plastic deformation. This deformation is not fully reccverable as the gyrating arm continues revolving and new inter-particle contacts are stressed. In this way, elastic energy is stored in the particles. Other research (26) indicates that some shales are subject to significant hysteresis, or incomplete stress recovery during cyclic loading.

When the shear stress reaches a certain magnitude, the particles crush or slide past one another. As the particles degrade and/or move with respect to one another part of the stored stored elastic energy is released, and the measured shear resistance appears to decrease (52). Particle movement stops and the shear stress must again increase to the magnitude of the static friction coefficient to induce further rearrangement. Alternatively, the shear stress must increase to the magnitude of the shale particles' shear strength to induce further degradation. The sample deformation is due to both particle degradation and rearrangement. This reaction sequence, called the "stick-slip" mechanism, has been found to occur in several types of shear strength tests (52).

In addition to the stick-slip mechanism, it may be that the hysteretic load response of the shale is independently responsible for some of the variation in the measured gyratory shear resistance. As mentioned above, the gyratory sequence causes the shale particle contacts to be loaded and unloaded in a cyclic manner. The hysteretic properties of the inter-particle contacts (i.e. incomplete elastic recovery) may be reflected in the overall behavior of the sample during cyclic loading.

Regression analyses were performed using the parameters listed in Table 9 for all gyratory compaction samples except the air roller tests which were carried out to 500 revolutions. The results of compaction using the oil and air rollers were considered independently. Using sample populations of 12 and 27, for the air and oil roller tests respectively, minimum significant values of Pearson's coefficient "R" were determined to measure the correlations between pairs of variables.

The regression analyses indicated a strong correlation between the gyratory shear resistance and axial ram pressure for both rollers. This close correspondence between applied stress and shear resistance is shown in Figure 25. A correlation coefficient of 0.97 was obtained for the data from both rollers. The tests continued to 100 revolutions for the oil roller show a somewhat wider scatter, but very close overlap with the 50 revolution data shown in Figure 25.

Fairly good correlations were obtained for the compacted unit weight as a function of ram pressure. These data have already been presented in Figure 23. Correlation coefficients of 0.79 and 0.72 were determined for the oil and air rollers respectively.

Table 9.

Gyratory Compaction Variables Used In Regression Analyses.

Symbol	Description	Units
RP	Axial Ram Pressure	psi
RN	Number of Gyratory Revolutions	
G_s	Gyratory Shear Resistance	psi
w	Sample Moisture Content	%
DD	Compacted Dry Unit Weight	pcf
IC	Index of Crushing	%
AA	Arithmetic Change in AGM	
PA	Percent Change in AGM	%

Also Considered Were The Computed Variables, Log_{10} of G_s, DD, IC, AA, PA.

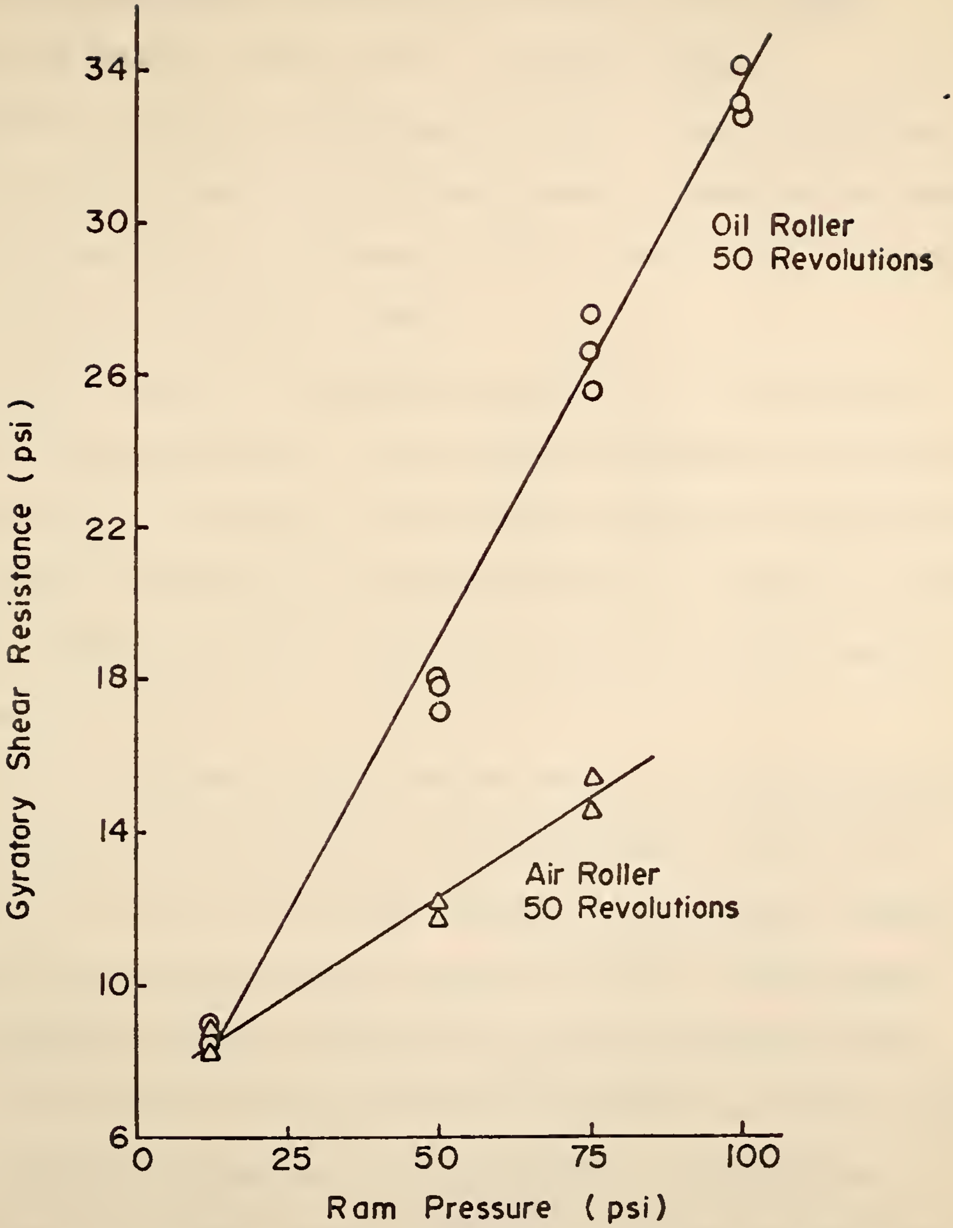


FIGURE 25 RELATION BETWEEN GYRATORY SHEAR RESISTANCE AND RAM PRESSURE FOR AIR AND OIL ROLLERS.

From these results it appears that the shale samples' shear resistance should correspond fairly well to the compacted unit weights. The air roller data showed an arithmetic relation between shear resistance and density, with $R = 0.74$. A somewhat better correspondence was found in comparing the change in the logarithm of the shear resistance ($\text{Log}_{10}\ G_s$) with the change in compacted unit weight. This relation resulted in a correlation of $R = 0.82$ for the air roller, and $R = 0.81$ for the oil roller data.

At present there is no recorded experience in the literature relating the compaction effort of the gyratory apparatus to the field compaction of shale embankments. Thus for the purposes of this study, gyratory shear resistance and compaction effort have little value unless they can be related to the more intrinsic parameters of field density and degradation.

Figure 26 shows the relation between dry unit weight and gyratory shear resistance for four tests performed with the air roller and an axial pressure of 50 psi. These data indicate a strong trend towards increasing shear resistance with increases in density. Furthermore the data show that for the air roller and a limited moisture range, there is less scatter in the gyratory shear resistance at higher density levels. This smaller range in shear resistance at high densities (which correspond to high compaction effort) is not obvious when the data are presented as in Figure 24. If this information reflects field compaction characteristics for the same or similar materials, significant design savings for shale embankments could be based on the assurance of uniform shear strength at particular compacted densities.

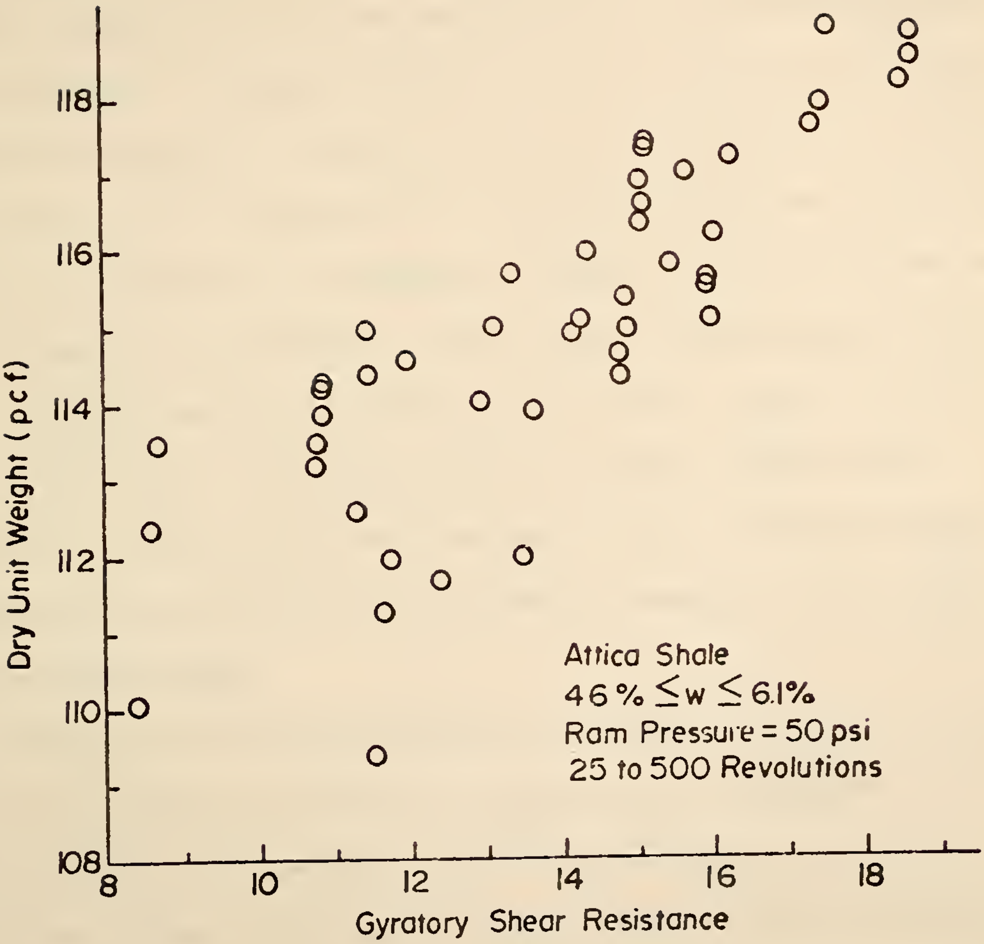


FIGURE 26 RELATION BETWEEN GYRATORY SHEAR RESISTANCE AND COMPACTED UNIT WEIGHT, USING THE AIR ROLLER.

However, more work will be needed before the gyratory compaction apparatus, with either the air or oil roller, can be relied upon for shear testing of shales to be used in compacted embankments.

Unfortunately the regression analyses did not yield particularly good correlations for the particle degradation parameters. For the oil roller, a value of R = 0.58 was determined for the relation between the logarithm of the shear resistance and the logarithm of the index of crushing ($\text{Log}_{10}\ G_s$ vs. Log_{10} IC). No other significant relations were found. On the other hand, the air roller data showed fairly good correlations between the compacted unit weight and the logarithms of both the percent and arithmetic change in AGM. The absolute values of the correlation coefficients were 0.82 for DD vs. Log_{10} PA, and 0.72 for DD vs. Log_{10} AA. Figure 27 shows the percent change in AGM (plotted to a logarithmic scale) and the compacted dry unit weights obtained using both rollers over the entire compaction effort range that was investigated.

Table 10 shows the significant correlation coefficients obtained from the regressions performed on single pairs of variables. It should be noted that the sample water content appears to have had a significant effect on the results of the air roller tests but not on the oil roller tests. All of the samples tested were within a fairly narrow moisture content range, as described for the kneading compaction tests. Specific values of the moisture content of the gyratory compaction samples were within these limits:

Oil Roller Samples: average water content = 4.2%
standard deviation = ± 0.6%

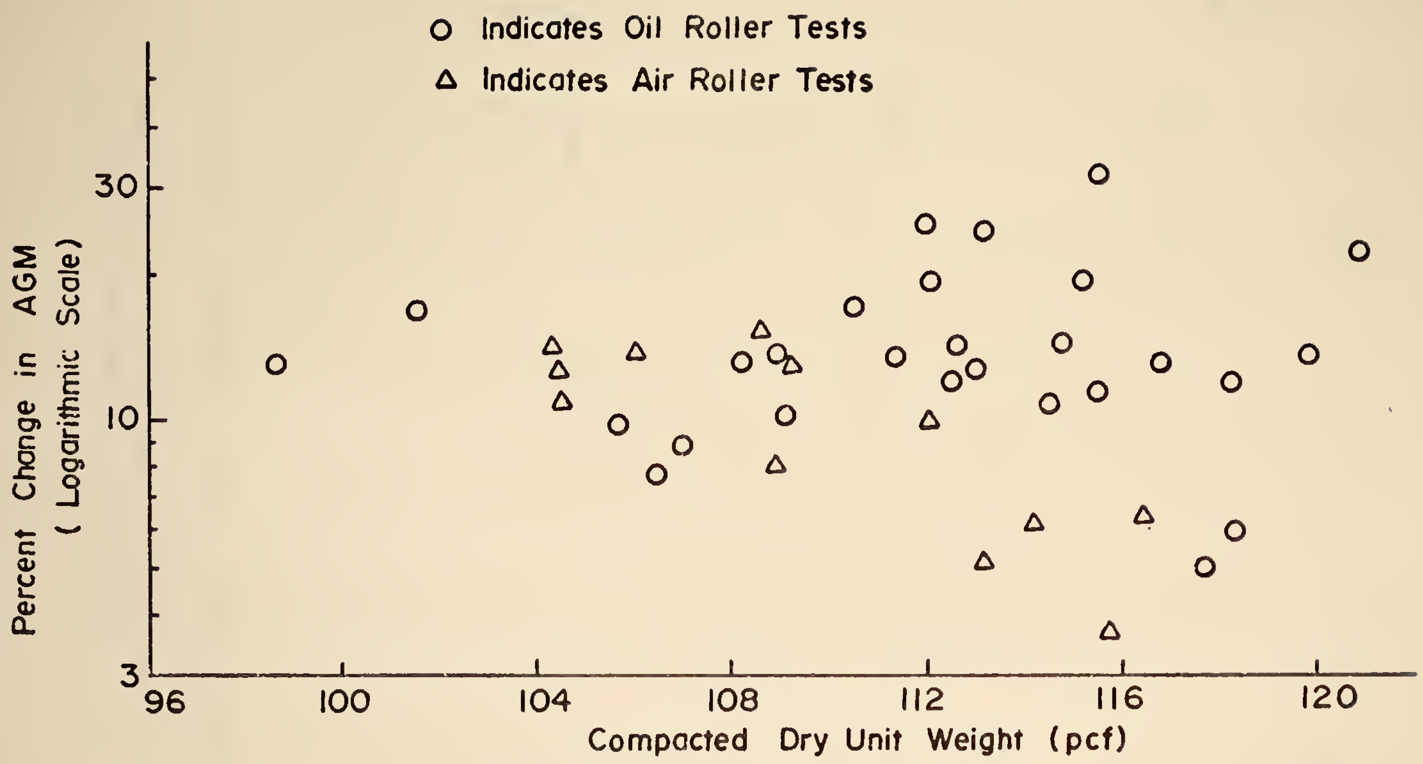


FIGURE 27 RELATION BETWEEN COMPACTED UNIT WEIGHT AND PERCENT CHANGE IN AGGREGATE GRADATION MODULUS.

Table 10.

Absolute Values of Significant Correlation Coefficients For Pairs of Gyratory Compaction Variables.

	G_s	DD	IC	AA	PA	Log_{10} G_s	Log_{10} DD
Oil Roller:							
RP	0.97	0.79	*	*	*	0.94	0.79
RN	*	0.58	*	*	*	*	0.57
G_s	-	0.80	*	*	*	-	0.80
w	*	*	*	*	*	*	*
DD	0.80	-	*	*	*	0.81	-
IC	*	*	-	-	-	0.51	*
AA	*	*	-	-	-	*	*
PA	*	*	-	-	-	*	*
Log_{10} G_s	-	0.81	0.51	*	*	-	0.81
Log_{10} DD	0.80	-	*	*	*	0.81	-
Log_{10} IC	0.51	*	-	-	-	0.58	0.58
Log_{10} AA	*	*	-	-	-	*	*
Log_{10} PA	*	*	-	-	-	*	*

- Indicates No Rational Basis For Correlation

* Indicates No Significant Correlation, i.e. $|R_{oil}| \leq 0.49$, and $|R_{air}| \leq 0.71$

Table 10 Continued.

	G_s	DD	IC	AA	PA	Log_{10} G_s	Log_{10} DD
Air Roller:							
RP	0.97	0.72	*	*	*	0.96	0.71
RN	*	*	*	*	*	*	*
G_s	-	0.74	*	*	*	-	0.73
w	0.72	0.78	*	*	*	0.74	0.78
DD	0.74	-	*	*	0.81	0.72	-
IC	*	*	-	-	-	*	*
AA	*	*	-	-	-	*	*
PA	*	0.81	-	-	-	*	0.80
Log_{10} G_s	-	0.72	*	*	*	-	0.71
Log_{10} DD	0.73	-	*	*	0.80	0.71	-
Log_{10} IC	*	*	-	-	-	*	*
Log_{10} AA	*	0.72	-	-	-	*	0.72
Log_{10} PA	*	0.81	-	-	-	*	0.81

- Indicates No Rational Basis For Correlation

* Indicates No Significant Correlation, i.e.

$|R_{oil}| \le 0.49$, and $|R_{air}| \le 0.71$

Air Roller Samples: average water content = 4.4%
standard deviation = ± 0.3%

Nevertheless, water content showed significant correlations with Log_{10} DD ($|R| = 0.78$), Log_{10} G_s ($|R| = 0.74$), and Log_{10} PA ($|R| = 0.71$) for the results of tests with the air roller. No significant correlations between moisture content and any other sample parameters were obtained for the oil roller data. No apparent reason for this discrepancy has been found.

Finally, multiple variable regression analyses were performed for the gyratory compaction tests, in the same manner as described in Section 4.1.3.

Considering the development of gyratory shear resistance as a function of compaction effort, the following equation was developed for the oil roller:

$$G_s = -0.814 + 0.0304(RN) + 0.326(RP) \qquad R^2 = 0.94$$

and for the air roller:

$$G_s = 5.22 + 0.00592(RN) + 0.128(RP) \qquad R^2 = 0.94$$

In neither case was the correlation improved by considering the effects of compacted unit weight or by using a logarithmic form of G_s.

Equations were also developed to determine the compacted unit weight as a function of the compaction effort. For the oil roller, the best relation was:

$$DD = 93.2 + 0.0700(RN) + 0.144(RP) + 1.27(w) \qquad R^2 = 0.79$$

and for the air roller:

$$DD = 80.1 + 0.0516(RN) + 0.163(RP) + 4.33(w) \qquad R^2 = 0.77$$

In both of the above equations, the sample moisture content was found to be the least significant term. Correlation was not improved by considering the logarithm of the dry unit weight.

Regressions performed on the oil roller data with particle dagradation as the dependent variable were found to produce the lowest correlations of all the regressions attempted. No significant relation could be found between the index of crushing and the other variables. The best degradation equation that could be produced was:

$$AA = 1.13 + 9.09 \times 10^{-4}(RN) - 6.58 \times 10^{-3}(RP) - 8.37 \times 10^{-3} + 0.0290(G_s) \qquad R^2 = 0.35$$

Regressions including the PA term were found to have slightly lower correlations than the AA dependent equations.

Regressions for degradation of the air roller samples gave slightly better results than those described above. The best relations were found to be:

$$PA = 60.7 - 0.0506(RN) - 0.0300(RP) - 0.400(DD) - 0.234(G_s) \qquad R^2 = 0.73$$

and:

$$PA = 27.4 - 0.0726(RN) - 0.125(RP) - 1.74(w) \qquad R^2 = 0.68$$

It is interesting to note that significant degradation predicting equations could be generated using the IC term for the air roller data, but both the IC and AA dependent equations had lower correlations than those based on the PA term.

4.2.5 Conclusions

Gyratory compaction tests using both the air and oil rollers resulted in comparable patterns for both densification and degradation of samples of the model shale.

Although initially it appeared that there was little similarity in the development of shear resistance between comparable tests, this appears to be a result of nonlinear stress-strain behavior of the type characterized as "stick-slip" in other types of strength tests on granular materials. Regression analyses of the test data resulted in a strong correlation between the samples' gyratory shear resistance and the axial ram pressure. A somewhat lower degree of correlation was found between the compacted dry unit weight and the ram pressure.

The compaction effort variable "number of gyratory revolutions" does show some correspondence to the compacted unit weight. Qualitatively, densification occurs at a decreasing rate as the number of revolutions becomes large. However regression analyses did not produce significant correlations between the number of revolutions and the gyratory shear resistance, compacted unit weight, or any of the three degradation parameters.

Correlations of particle degradation with either compaction effort or gyratory shear resistance were found to have little or no significance. Correlations between sample density and degradation were somewhat better.

Regression equations were developed to relate the various parameters investigated. These equations may be useful in planning

future test programs, comparing the gyratory test with other laboratory compaction methods, or in laboratory testing of materials for embankment construction.

It appears that much work needs to be done before the gyratory compaction test can be used for shear strength testing of compacted shales. Additionally gyratory compaction appears to have little value as an index test for evaluating shale particle degradation. The development of such an index is considered elsewhere in this report.

4.3 Static Compaction

4.3.1 Introduction

Static compaction tests have been used in Great Britian to measure the strength of aggregates for highway construction since before 1947 (54). The British Standards Institution (BSI) has established a test for the determination of an "aggregate crushing value" which uses a static load to cause degradation in aggregate samples (11). Experience in South Africa shows this test to be useful for most road building materials, but like the LA Abrasion Test, it is relatively insensitive to strength differences in shales and other "weak" rocks (54).

The BSI aggregate crushing test measures the amount of fines produced from application of a 40 ton load to an aggregate specimen confined in a rigid steel mold. The initial sample consists of particles which pass a 12.70 mm sieve and are retained on a 2.40 mm sieve. Following the test, sample fines are considered to be the particles which pass the 2.40 mm sieve. Degradation is defined as the

ratio of fines to the total sample weight, expressed as a percentage.

The 10% Fines, Aggregate Crushing Test (10-FACT) was developed in 1959 by Shergold and Hosking to avoid some of the problems associated with the BSI aggregate crushing test (76). Specifically, the 10-FACT enables very weak rock materials to be tested by considering the applied load to be a variable for each material. The 10-FACT characterizes materials on the basis of the load in tons required to produce 10% fines, using the same sample and apparatus specifications as the BSI test (76).

The 10-FACT has been used to compare strength, compacted density, and changes in void ratio for aggregate samples. Loubser has reported correlations between results of the 10-FACT and compacted unit weight and water absorption for shales used in highway construction (54). On the basis of three highway performance case histories, Loubser concludes, "The crushing value (10-FACT) will indicate to what degree such foundation material as shale will deteriorate during construction or, later, under traffic and which techniques are advisable to make such a material perform satisfactorily".

4.3.2 Test Procedure

The test procedure used in this investigation for static compaction tests was adapted from the 10-FACT procedure described by Shergold and Hosking (76). Tests were performed on samples of Shale No. 11 which weighed between 1.5 and 2.0 pounds each. Sample size, or weight, did not appear to affect the test results. The samples initially were of the "A" gradation previously described, and the IC,

AA and PA degradation parameters were used to measure the degradation which occurred.

Fifteen of the tests were performed by placing the crushed shale samples in plastic sample bags in an open-ended cylindrical steel mold in the manner described for the gyratory compaction tests. The sample mold was 1/4 inch thick steel pipe which had a diameter of 4.00 inches and a height of about 5.5 inches. The sample and mold were placed on a steel plate, and a steel loading ram was inserted in the top of the mold so that it rested directly on the sample. The loading configuration is shown in Figure 28.

The sample, mold, base plate and loading ram were placed in a compressional testing machine and a vertical load was applied to the top of the loading ram. Samples were tested using applied loads of 10, 25, 35, 50, 75, 100, 150, and 250 psi. The load was maintained until visible movement of the dial gauge (attached to the loading ram) had stopped. The gauge was readable to the nearest 0.001 inch. Dial gauge readings before and after loading enabled calculation of the initial and final sample unit weights.

The other static compaction tests differed only in the method of placing the sample in the mold. Following the procedure for the BSI aggregate crushing test, the samples were placed in the mold in three layers. Each layer was tamped gently 25 times with a steel rod, one half inch in diameter.

A third group of tests also provided compacted unit weight data for static compaction tests. The sample height was measured for most of the gyratory compaction samples after the vertical load was applied

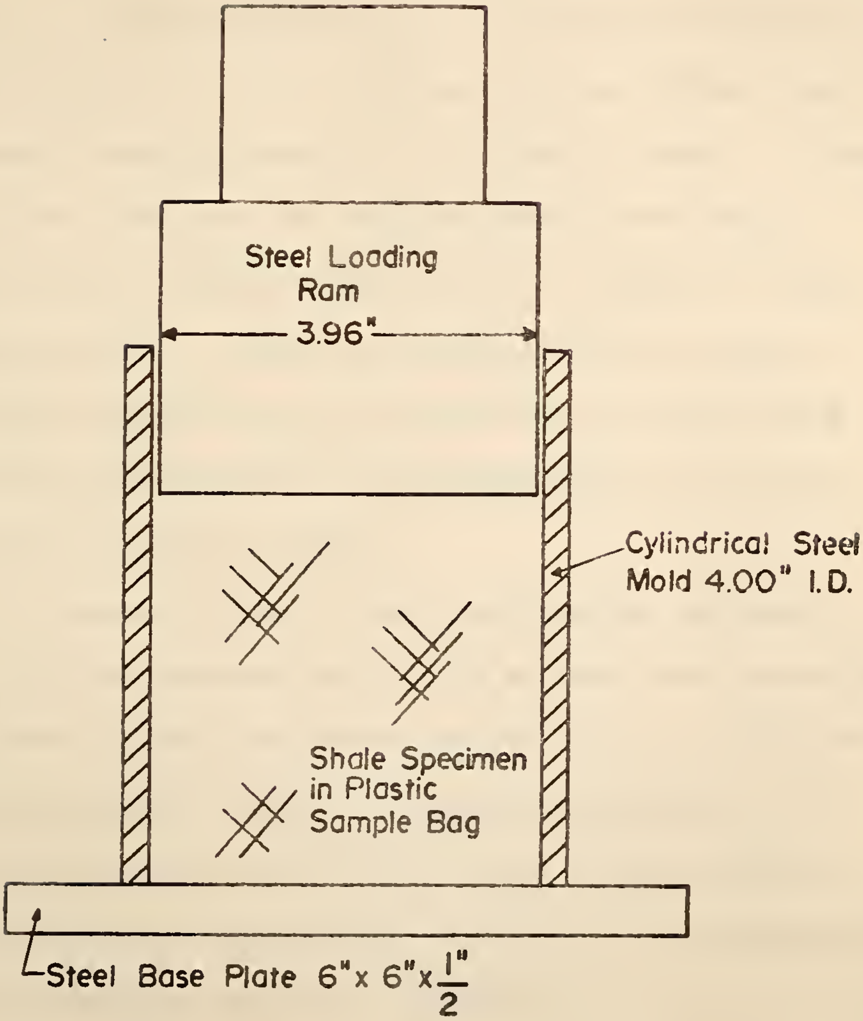

Steel Loading Ram
3.96"
Cylindrical Steel Mold 4.00" I.D.
Shale Specimen in Plastic Sample Bag
Steel Base Plate 6"x 6"x 1/2"

FIGURE 28 STATIC COMPACTION SAMPLE READY FOR TESTING.

but before gyration was started. This provided essentially the same loading mechanism as with the compressional testing machine, except that on the gyratory apparatus the ram moved up to confine the sample from below.

4.3.3 Test Results and Analyses

The results of static compaction tests on the Attica model shale are tabulated in Appendix D. Twenty-three samples were tested, eight of which were tamped during placement in the compaction mold. In general the results of these tests were very self-consistent, even though a relatively small number of samples were tested for a relatively large range in compaction effort. Further corroboration of the confined crushing test results was provided by the compacted unit weight data resulting from initial confinement of 61 gyratory compaction samples, before the gyratory process was started.

Computerized regression analyses were performed on the static compaction data with the compacted unit weight and the three degradation parameters considered as functions of the applied load, P. Additional correlations were attempted between the degradation parameters, the compacted unit weight, and the logarithms of these terms. Absolute values of the significant "R" correlation coefficients from regressions on single pairs of variables are shown in Table 11.

Of the three degradation parameters, only the index of crushing was found to have a significant correlation with the applied load. Although the AA and PA terms could be related to the compacted unit weight, the resulting correlation coefficients had lower values than

Table 11.

Absolute Values of Significant Correlation Coefficients For Pairs of Static Compaction Variables.

	P	DM	Log_{10} DM
Samples Not Tamped During Placement In Mold:			
IC	0.89	0.96	0.95
AA	*	0.89	0.89
PA	*	0.84	0.84
DM	0.93	-	-
Log_{10} IC	0.83	0.94	0.94
Log_{10} AA	*	*	*
Log_{10} PA	*	*	*
Log_{10} DM	0.92	-	-
Samples Tamped During Placement In Mold:			
IC	0.98	0.85	0.84
AA	*	*	*
PA	*	*	*
DM	0.88	-	-
Log_{10} IC	0.96	0.88	0.87
Log_{10} AA	*	*	*
Log_{10} PA	*	*	*
Log_{10} DM	0.87	-	-

- Indicates No Rational Basis For Correlation

* Indicates No Significant Correlation, i.e. Tamped Samples: $|R_{DM}| \leq 0.83$, and $|R| \leq 0.64$ For Degradation; Samples Not Tamped: $|R| \leq 0.64$

that developed for the IC term.

The regressions demonstrated an extremely good linear relation between the compacted unit weight and the index of crushing, and the compacted unit weight and applied load. The close correlation between these three terms indicates that the scatter in the AGM data is not the result of poor testing, but suggests that the two AGM terms may not be appropriate for measuring particle degradation due static compaction.

Table 11 also shows the results of correlations between variable pairs for the samples that were placed and tamped in three layers before the compaction load was applied. Neither of the AGM terms proved to be significant for these tests. Figures 29 and 30 show the relation between the index of crushing and ram pressure for the two groups of samples.

Several equations were developed to describe the data shown in Figures 29 and 30. For the untamped samples the best relation was:

$$IC = 4.10 + 0.093(P) \qquad R^2 = 0.79$$

and for the tamped samples:

$$IC = 6.60 + 0.049(P) \qquad R^2 = 0.95$$

The samples which were not tamped when placed in the static compaction mold were also subjected to a slightly different form of analysis. Since it could logically be expected that the amount of degradation would approach zero for very low magnitudes of applied load, the "best" equation to predict the particle degradation from static compaction should indicate this. A regression analysis was performed which forced the generation of an equation through the origin of Figure 29.

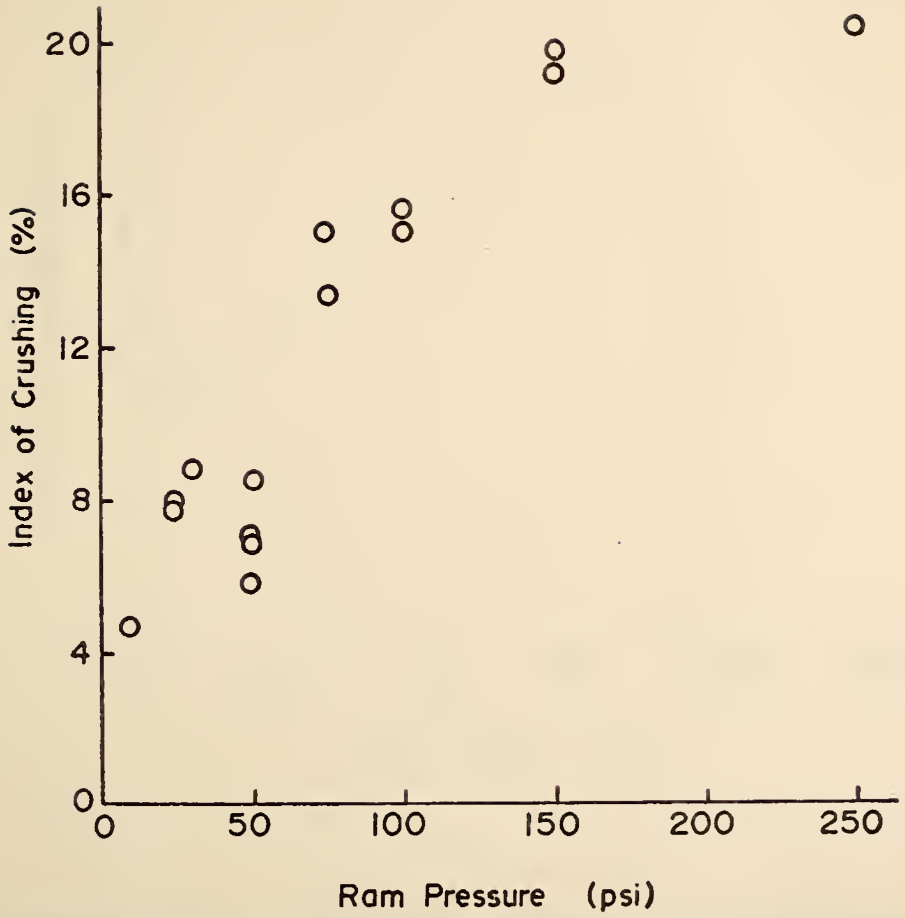


FIGURE 29 PARTICLE DEGRADATION FROM STATIC COMPACTION TESTS ON UNTAMPED SAMPLES.

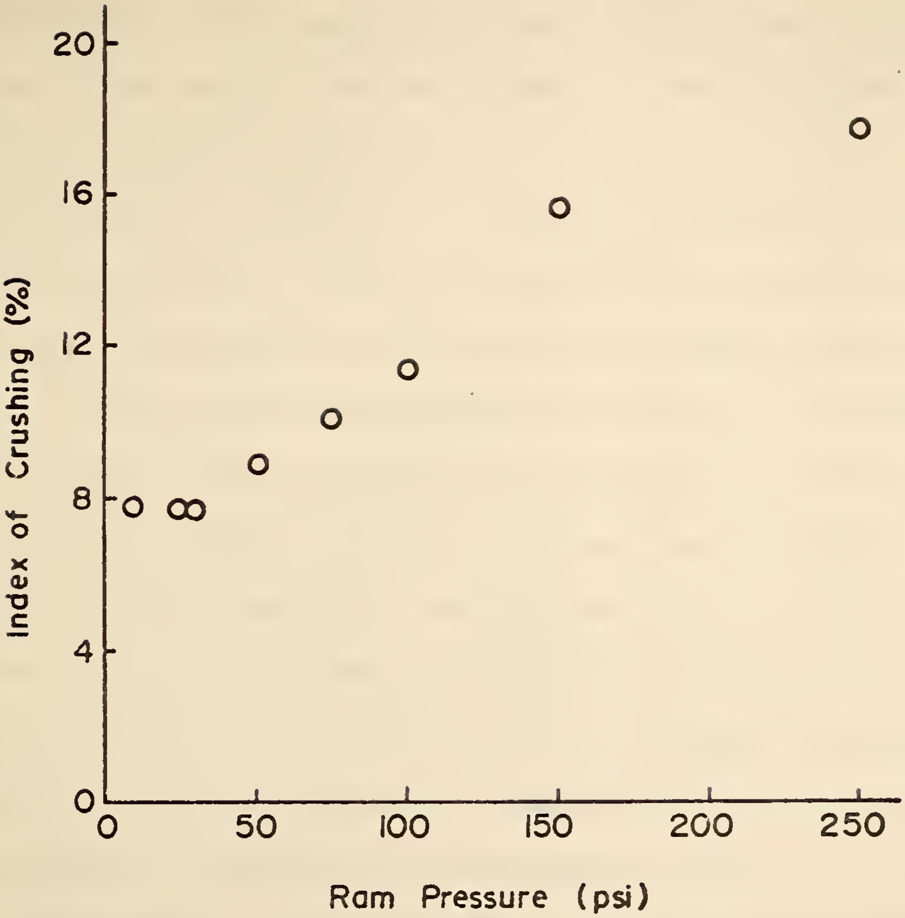


FIGURE 30 PARTICLE DEGRADATION FROM STATIC COMPACTION TESTS ON TAMPED SAMPLES.

The equation which resulted was:

$$IC = 0.133(P) \qquad R^2 = 0.71$$

which has a somewhat lower degree of correlation than the equation described above. This suggests that there is more than a simple linear relation between applied load and degradation, and a qualitative inspection of Figure 29 confirms this. Both Figures 29 and 30 show that the degradation rate decreases with increasing compaction effort.

Regressions which forced the generation of an equation through the origin were not performed on the data from the samples which were tamped in three layers during placement in the mold. The tamping may be considered as a form of impact compaction, which may have caused some degradation of the shale particles before any load was applied. As shown in Figure 30, the IC values for ram pressures of 10, 25 and 35 psi were all about 7.7%. The magnitude of degradation increased only when the applied load was 50 psi or greater. This may indicate a degradation threshold due to the precompaction (tamping), however not enough tests were performed to provide conclusive evidence of this.

Equations were also developed to describe the compacted unit weight resulting from static compaction. For the samples which were not tamped, the best relation was:

$$DM = 87.8 + 0.172(P) \qquad R^2 = 0.87$$

and for the tamped samples:

$$DM = 96.8 + 0.125(P) \qquad R^2 = 0.77$$

A similar analysis for the initial compacted unit weights of 61 gyratory samples resulted in the following equation.

$$DD = 73.6 + 0.337 \qquad R^2 = 0.53$$

Figure 31 shows the average compacted unit weight values obtained from gyratory compaction samples following load application but before gyration was started. These data show somewhat greater scatter (i.e. a lower value of the correlation coefficient) however a clear trend in the mean values of compacted unit weight for the various levels of applied load can be seen in Figure 31.

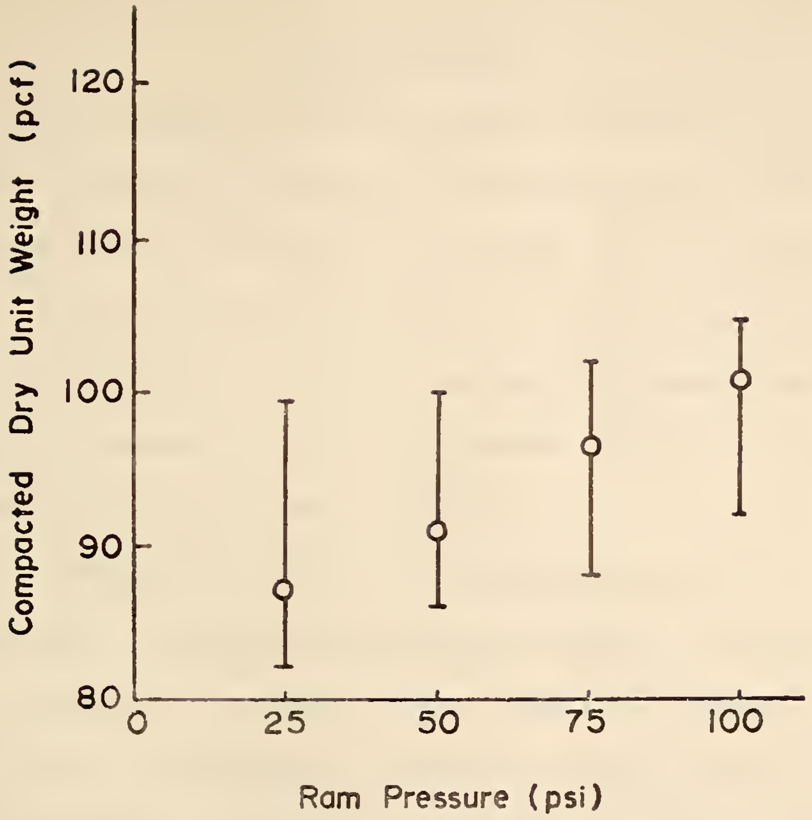
During the static compaction tests, several minutes were required before the sample deformation reached a maximum. This time period was not constant, in fact several of the samples continued to deform slowly over periods of up to about 30 minutes. This waiting period was not fully observed between load application and the start of gyratory compaction, with the result that the data shown in Figure 31 may be considered less precise than the data shown in Figures 29 and 30. The results of static compaction with the gyratory apparatus are presented here because the trend is not dissimilar to, and provides some corroboration of, the more carefully controlled tests performed with the compressional testing machine.

Finally, multiple variable regression analyses were performed on the results of static compaction tests (not including those performed with the gyratory apparatus) to determine the relation between the applied load, compacted unit weight and particle degradation. Despite the poor correlation between the arithmetic change in aggregate gradation modulus and the applied load and unit weight terms considered individually, the best relation for the untamped samples was:

$$AA = 1.17 + 3.28 \times 10^{-3}(P) - 1.28 \times 10^{-2}(DM) \qquad R^2 = 0.90$$

For the tamped samples, the best relation was:

$$IC = 15.14 + 5.78 \times 10^{-2}(P) - 8.90 \times 10^{-2}(DM) \qquad R^2 = 0.96$$



○ Indicates Mean Value for Range Shown

FIGURE 31 AVERAGE COMPACTED UNIT WEIGHTS FROM STATIC COMPACTION TESTS USING GYRATORY APPARATUS.

The high values of the correlation coefficients for these two equations indicate that these relations can be used to predict the degradation resulting from static compaction tests with a great deal of accuracy.

4.3.4 Conclusions

The static compaction test procedure which was used in this investigation resulted in very good correlations between the compaction effort, compacted unit weight and the IC particle degradation variable. The correlations appeared to be slightly improved for regressions performed on the data from the eight samples which were tamped during placement in the mold, however the use of a plastic sample bag simplifies the test procedure.

Static compaction tests have been used successfully to evaluate the degradation of highway construction materials, including shales, in both Great Britain and South Africa. The test procedure developed and used in this investigation for the Attica model shale is fairly simple and requires no unusual or particularly sophisticated equipment. Further static compaction tests on a variety of Indiana shales could result in the development of a particle degradation index test of sufficient reliability to be applied to classification of shales for embankment construction.

4.4 Impact Compaction

4.4.1 Introduction

Impact compaction tests are a fundamental part of most soils investigations for embankment construction (88, 92). The current state of the art includes several "standard" test procedures (2, 3, 86,

88) which have developed from a method described by R. R. Proctor in 1933 (67). Impact compaction tests, loosely described as "Proctor compaction tests", have been used to investigate and define the densification of various soils as a function their moisture content and compaction effort. The test typically is performed in a four or six inch diameter mold with the compaction effort applied through blows of a freely falling compaction hammer of significantly smaller diameter than the mold. The various aspects of this test, including applicability to field compaction and embankment design, have been thoroughly discussed in the literature (46, 88, 92).

Another type of impact compaction test uses a relatively larger compaction hammer which closely fits the inside diameter of the compaction mold. This test has been used in England, where a standard method has been developed to indicate the strength or resistance to impact of aggregates (11).

A review of the literature found no previous work which attempted to relate particle degradation, for either type of impact compaction test, with degradation of materials used in embankment construction. Both types of impact compaction were slightly modified and used in this investigation to test the Attica model shale.

4.4.2 Particle Degradation, Full Face Compaction Test Results

Compaction-degradation tests were performed using a four inch diameter mold with a closely fitting compaction hammer. The compaction effort was applied by a ten pound drop weight which fell freely from a height of eighteen inches. The compaction effort was applied using an electricity powered automatic apparatus manufactured by the

Rainhart Co. The drop hammer is in compliance with specifications of the ASTM "Marshall test" for bituminous mixtures (5). Tests were performed using the "A" gradation, as previously described in Section 4.1.

Compaction tests were performed using three levels of compaction effort. Presieved samples of Shale No. 11 were compacted by 5, 10 or 20 blows each. The sample volume was determined by using the average of four height measurements, made after every five blows. The samples were again sieved at end of the compaction tests and IC and AGM values determined. Test results are tabulated in Appendix D.

Figure 32 shows final and intermediate unit weights which resulted from this form of impact compaction. The results indicate fairly good uniformity at the same levels of compaction effort.

Figure 33 and 34 show the results of particle degradation analysis on the ten samples tested. These figures clearly show that although the impact compaction method produced fairly uniform and repeatable levels of compacted unit weight, the particle degradation results varied widely.

The variation in particle degradation was not due to the effects of moisture. All of the samples tested had water contents between 4.2 and 4.5%. Nor was the variation due to differences in the initial particle gradation of the samples. Observations made during the tests show no reason for considering any of the test data as "bad" or unusual in comparison with the total results.

Observations made at the end of the impact compaction tests showed that a great deal of particle stratification existed within the samples

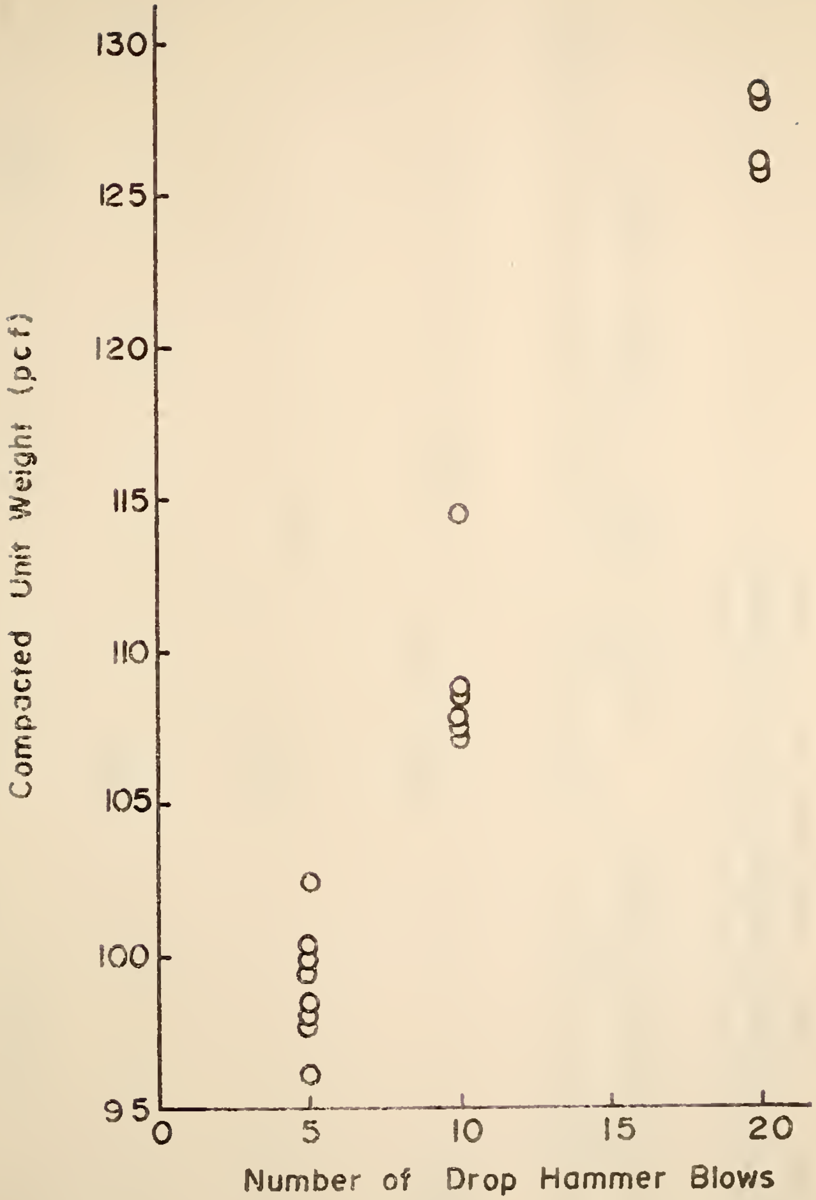


FIGURE 32 COMPACTED UNIT WEIGHTS RESULTING FROM FULL FACE IMPACT COMPACTION AT VARIOUS EFFORT LEVELS.

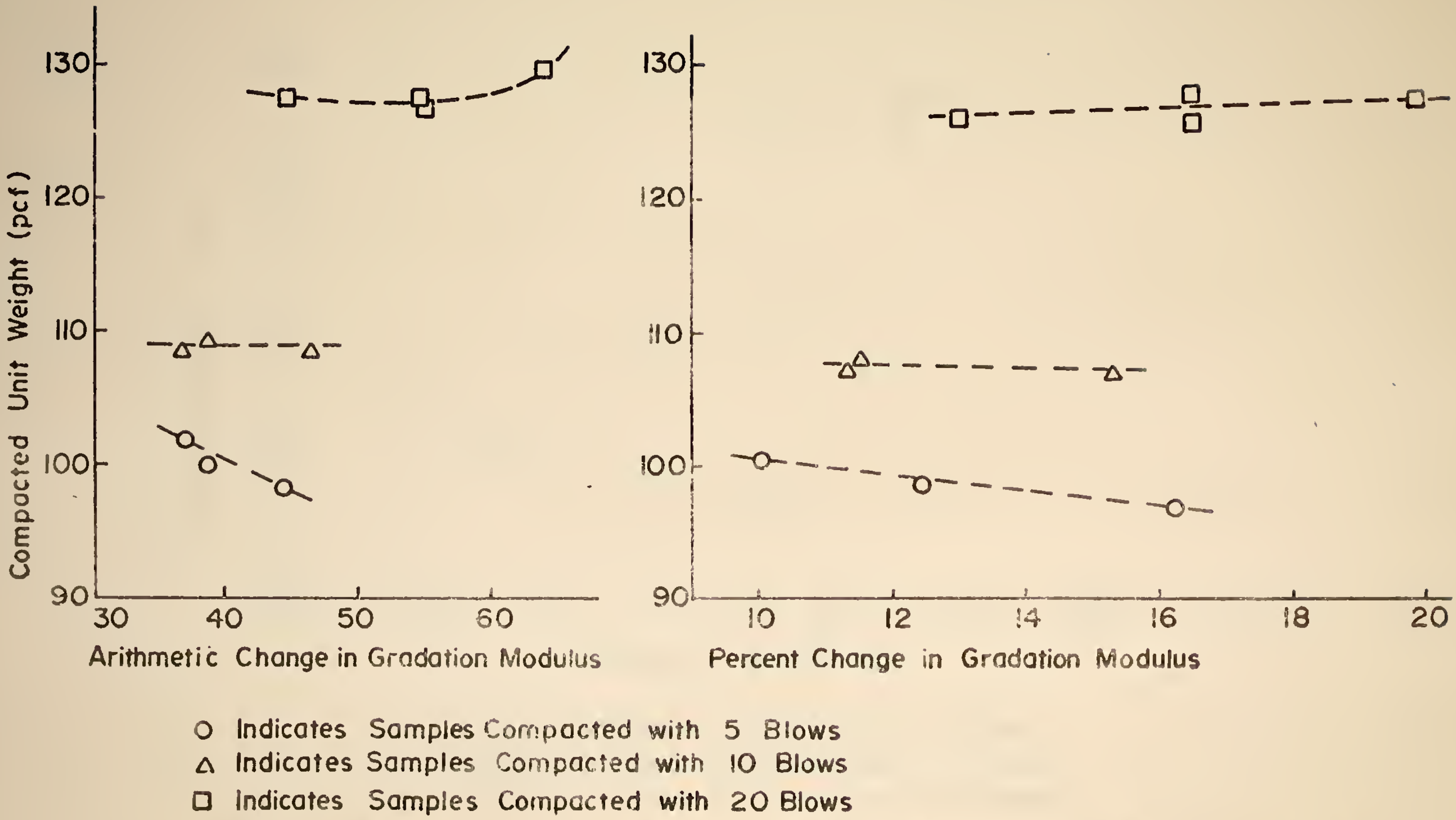


FIGURE 33 PARTICLE DEGRADATION (AGM) RESULTS FROM FULL FACE IMPACT COMPACTION TESTS.

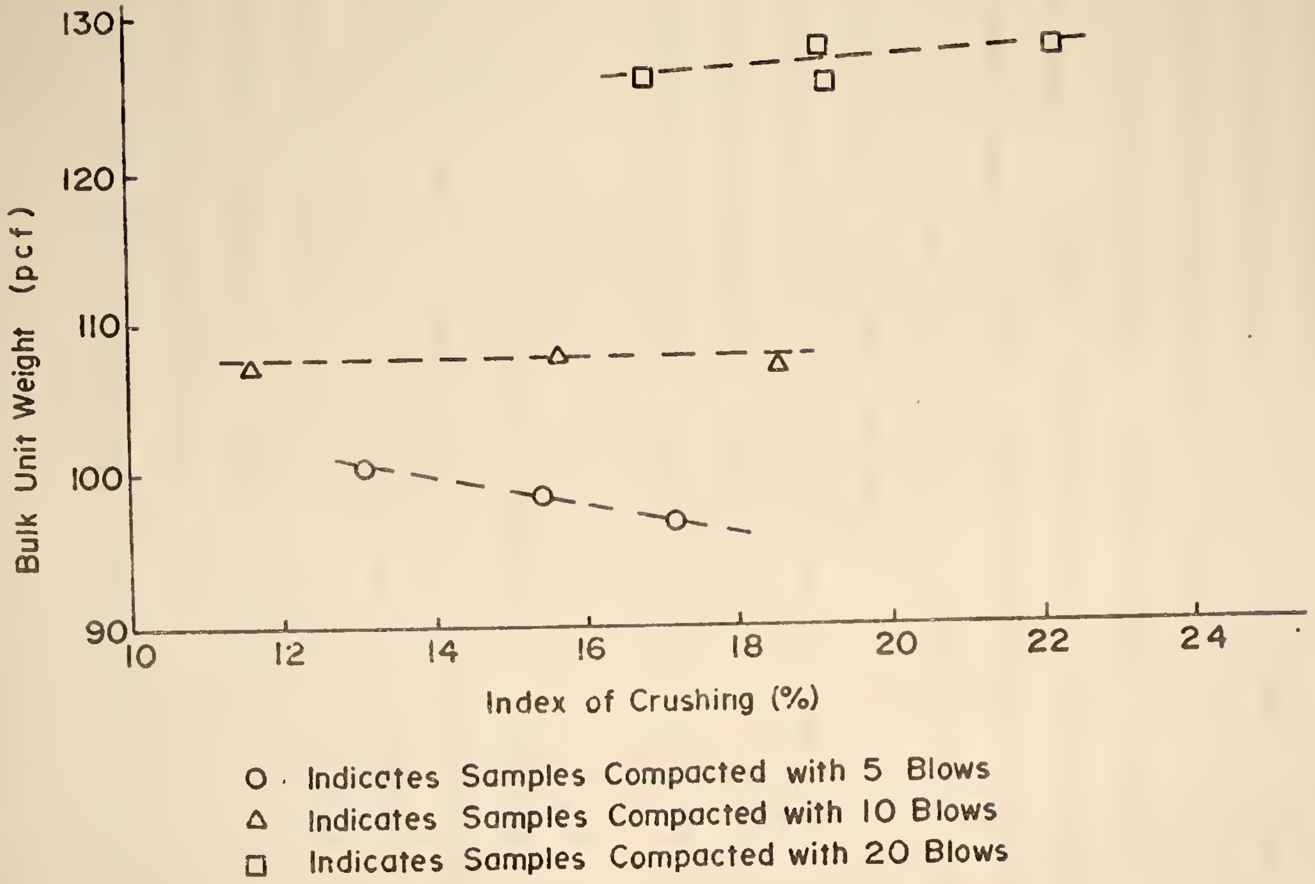


FIGURE 34 PARTICLE DEGRADATION (IC) RESULTS FROM FULL FACE IMPACT COMPACTION TESTS.

when they were extruded from the compaction mold. This stratification consisted of a large amount of fines in the upper portion of the samples, with relatively larger particles concentrated at the base of the samples. As some care had been taken to thoroughly mix the particles during initial placement in the mold, it appears that the drop hammer impact caused highly localized particle degradation at the top of the sample. It can by hypothesized that the first blow(s) of the drop hammer caused the shale particles to "lock up" in the upper portion of the sample, and that further compaction effort was not distributed uniformly throughout the sample.

The poor repeatability of particle degradation measurements for this type of impact compaction indicate that this test is unsuitable as an index for shales. Further tests were not performed for this reasons.

4.4.3 Moisture-Density, Proctor Compaction Test Results

Proctor type impact compaction tests have been used to prepare specifications for the use of shales in compacted embankments. Deo (23) has described the shale embankment compaction specifications used by three agencies (Indiana State Highway Commission; U. S. Soil Conservation Service; and the Division of Water, Indiana Department of Natural Resources), and their relation to the results of Proctor compaction tests.

Deo also reported the results of Proctor compaction tests using four levels of compaction effort on fifteen Indiana shales. These tests were performed in a six inch diameter mold, with a maximum particle size of 3/4 inch. While Deo did not specifically study the

effects of particle gradation or degradation, his report states that the optimum moisture content tended to increase as the percentage of sample fines (defined as material smaller than 0.005 mm) increased. Deo also noted that increased compaction effort caused higher densities at lower values of optimum moisture for individual shales, but that the values of OMC and $\gamma_{D_{max}}$ varied greatly between different types of shale.

Deo's shale compaction tests were performed immediately after mixing water with the crushed shale particles. His report does not mention the effects of allowing the moistened shale to "cure" as suggested in some test procedures (86, 88) nor does he discuss the repeatability of specific test results (23). Currently the Indiana State Highway Commission allows samples to cure for two days before testing (41).

Proctor type impact compaction tests performed in this investigation followed the method described by ASTM specification D698-64T, method C, with the following exceptions.

1. Tests were performed using both the standard 5.5 pound drop hammer and a 2.75 pound drop hammer. As the same compaction procedure was followed for tests using both hammers, a twofold difference in compaction effort was obtained.

2. Shale samples were wasted following completion of each test at a specific water content. The material was not reused because it was anticipated that particle degradation occurring during one compaction test might influence the results of subsequent tests.

Typically the Proctor compaction test is used to determine the

compacted dry unit weight which can be achieved at various moisture conditions for some constant level of compaction effort. Figure 35 is from tests conducted by the ISHC (41) and as part of this investigation.

The four compaction curves shown in Figure 35 represent series of tests performed on four different samples from the same shale formation. Ideally, curves B, C and D should be identical, as all of the test conditions were held constant. The displacement of these three curves from one another reflects variations in the compaction test results which can be considered due to the nonhomogeneous nature of the material. Variations in particle shape, size and strength probably cause the major part of the difference in test response.

Curve A represents compaction tests performed on samples with the "A" gradation previously described. The other curves in Figure 35 represent samples which had a smaller maximum particle size. If the results shown by curve D are overlooked, it appears that the presence or absence of particles between the 3/4 inch and No. 4 sieves has little effect on the magnitudes of maximum compacted dry unit weight and optimum moisture content. The variation in results between curve D and the other curves further emphasizes the already described need for a range of statistically validated measurements, rather than single values, for shale compaction parameters.

Figure 36 shows the compacted unit weight and degradation values resulting from six Proctor-type impact compaction tests. These tests were conducted using the 2.75 pound drop hammer after initial trials with the 5.5 pound hammer resulted in problems in removing the samples from the compaction mold without causing further degradation. This

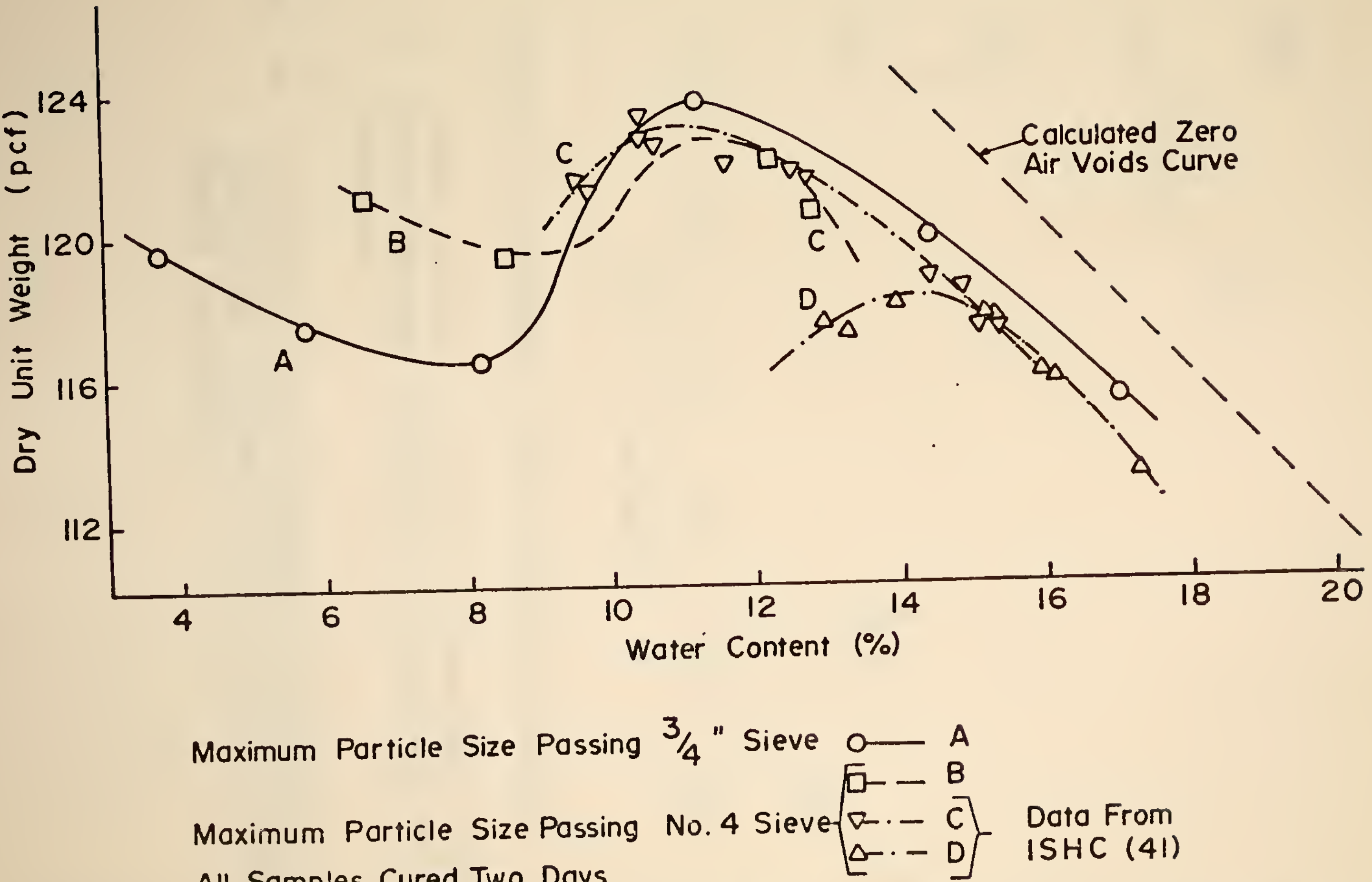
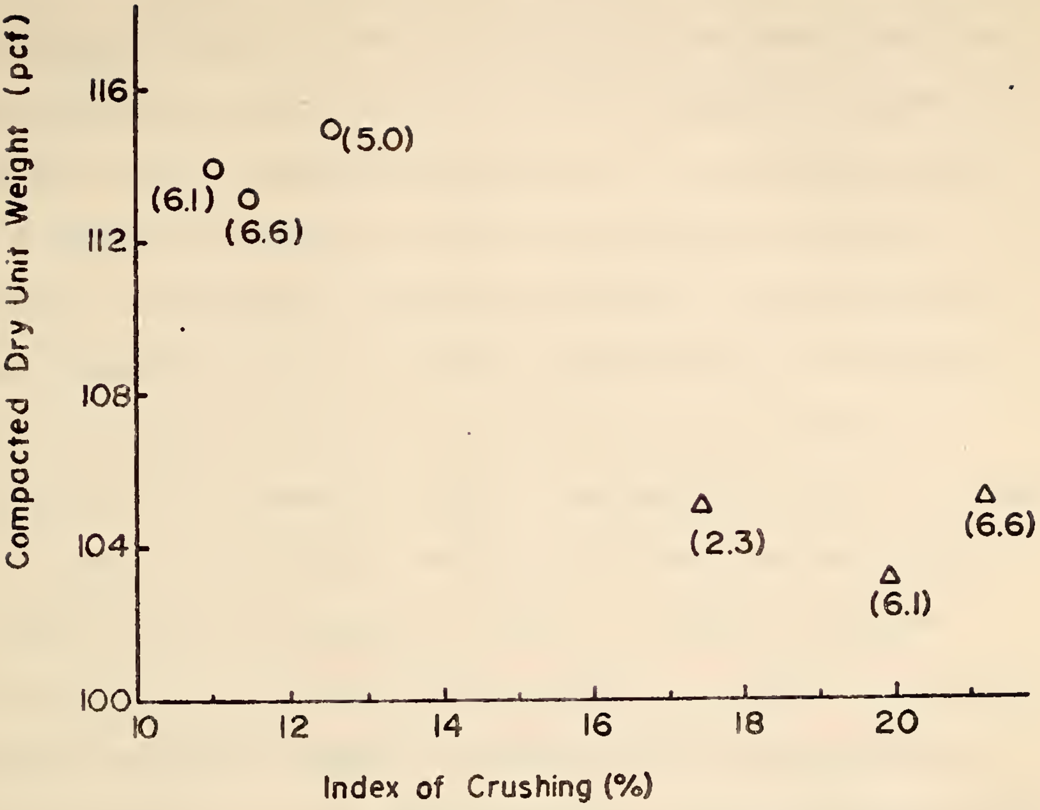


FIGURE 35 MOISTURE-DENSITY RESULTS FROM PROCTOR COMPACTION TESTS USING 5.5 LB DROP HAMMER.



Numbers in Parentheses Indicate Sample Water Content.

○ Indicates Samples Compacted at Original Water Content.

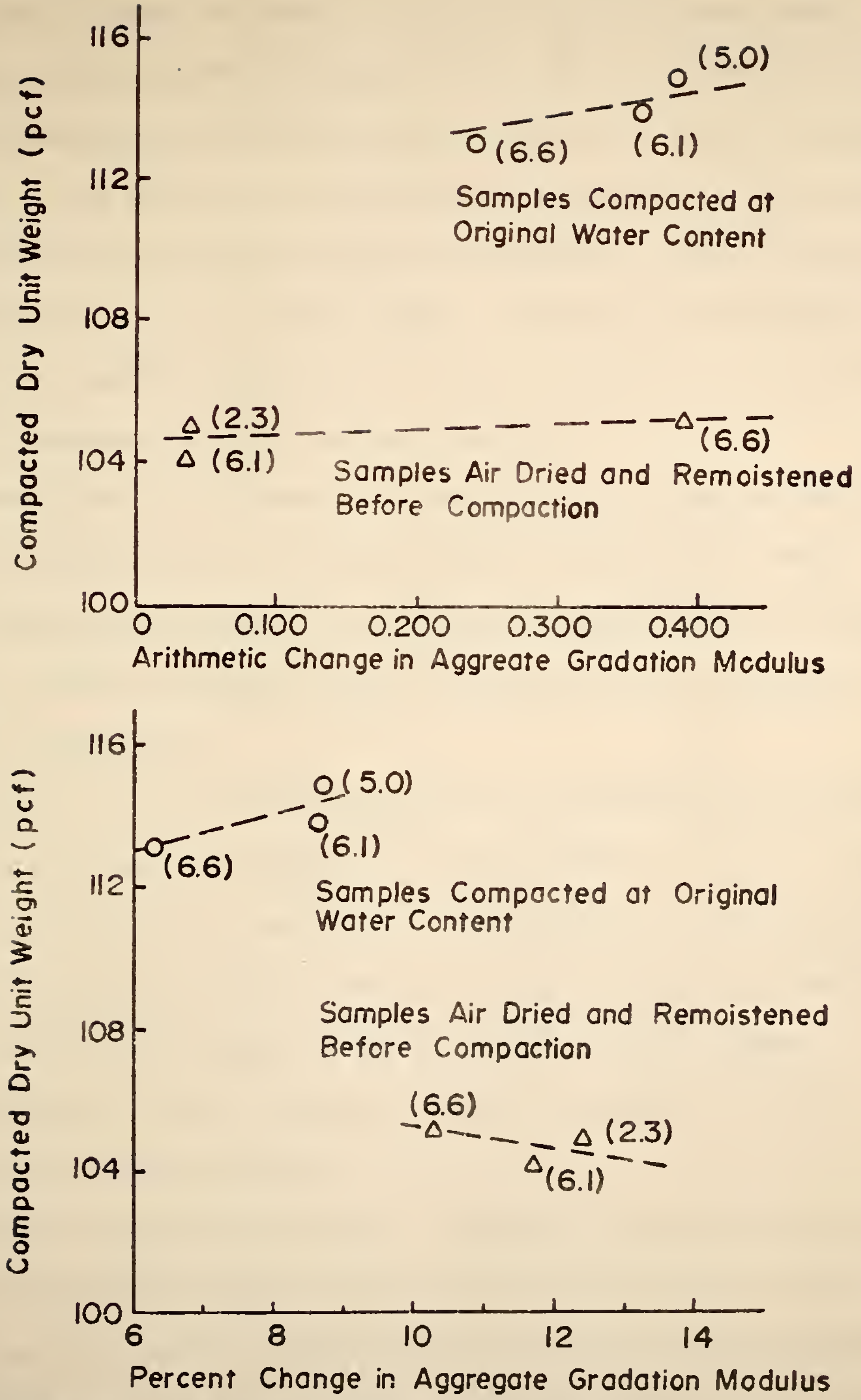
△ Indicates Samples Air Dried and Remoistened Before Compaction.

FIGURE 36 DENSITY-DEGRADATION (IC) RESULTS FROM PROCTOR COMPACTION USING 2.75 LB DROP HAMMER.

problem, which was encountered in all of the compaction tests, is discussed further in Appendix B. Problems of a different sort were also encountered in trying to analyze particle degradation using the lighter hammer. Figure 37 shows the degradation determined using the aggregate gradation modulus for the same tests presented in Figure 36. Both figures shows a considerable variation in the degradation parameters of samples with comparable densities and moisture contents.

Figure 36 also shows a significant difference in the degradation of samples compacted after the moisture content had been artifically changed. It appears that allowing the shale to dry and then be remoistened results in greater particle degradation (as measured by the IC and PA, but not the AA) and lower values of compacted unit weight. This suggested that sample moisture should be further manipulated and the effects noted, as part of this study. As a result, tests were initiated in which shale samples were mixed with varying amounts of water and allowed to cure for varying periods of time before compaction. This is in keeping with the conventional use of the Proctor compaction method.

Early tests showed that sample moisture contents in excess of 6% or 7% caused the finer shale particles to lump together and to coat the larger particles. While this tendency may be present for any amount of moisture in the shale, it was evident that artifically wetted shale samples could not be divided by mechanical sieving to enable determination of the degradation parameters. Further degradation analyses of shales compacted using the Proctor impact method were not performed for this reason.



Numbers in Parentheses Indicate Sample Water Content.

FIGURE 37 DENSITY-DEGRADATION (AGM) RESULTS FROM PROCTOR COMPACTION USING 2.75 LB DROP HAMMER.

Figure 38 shows the results of two series of Proctor impact compaction tests performed on samples with the "A" gradation, using a 5.5 pound drop hammer. The compaction samples were cured for either two days or ten days after the addition of moisture, and then tested. The water content of these samples was adjusted by using a water spray and gentle mixing with a small spatula. Curing was accomplished by placing the samples in steel pans, sealed in plastic, and storing them at room temperature for the required period.

From Figure 38 it appears that the eight day difference in curing time had little effect on the densification of the shale. This appears especially evident from the close correspondence of the samples tested at moisture contents of about 5.5% and 8%, but may also be entirely coincidental.

Figures 39 and 40 show the results of compaction tests performed using the 2.75 pound drop hammer and curing times of 0, 1, 2, 4 and 6 days. It appears that many of the samples cured for one day had slightly lower unit weights compared to those samples which were not cured. However the samples cured for two days tended to have slightly higher values of dry unit weight at comparable moisture contents, as shown in Figure 39.

Figure 40 does show one major difference in the compaction results for samples not cured and for samples cured for one or two days. The compaction curve which could be drawn through the points representing samples which were not cured, appears to have two peaks. This type of compaction curve, called "double peaked", is also evident in Figures 35 and 38 for samples cured two or ten days and compacted with the 5.5

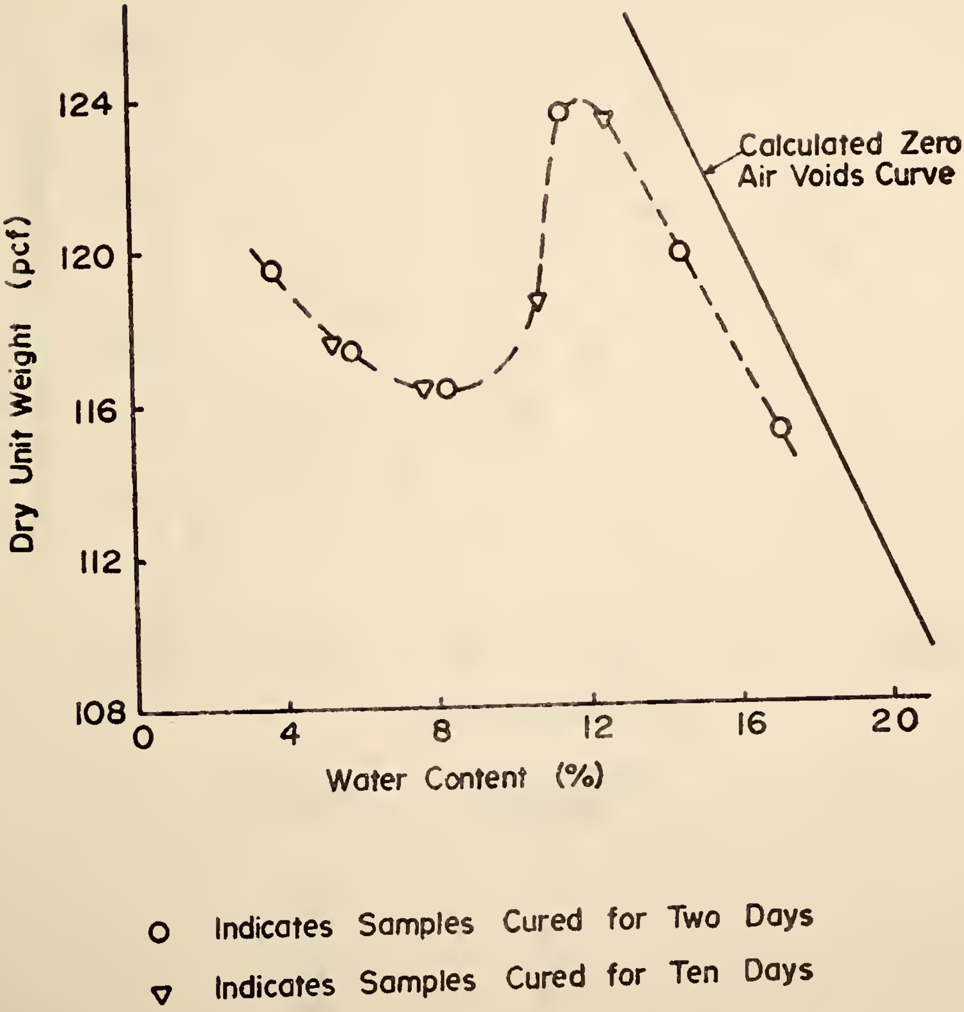


FIGURE 38 MOISTURE-DENSITY RESULTS FROM PROCTOR COMPACTION TESTS USING 5.5 LB DROP HAMMER FOR VARIOUS SAMPLE CURING TIMES.

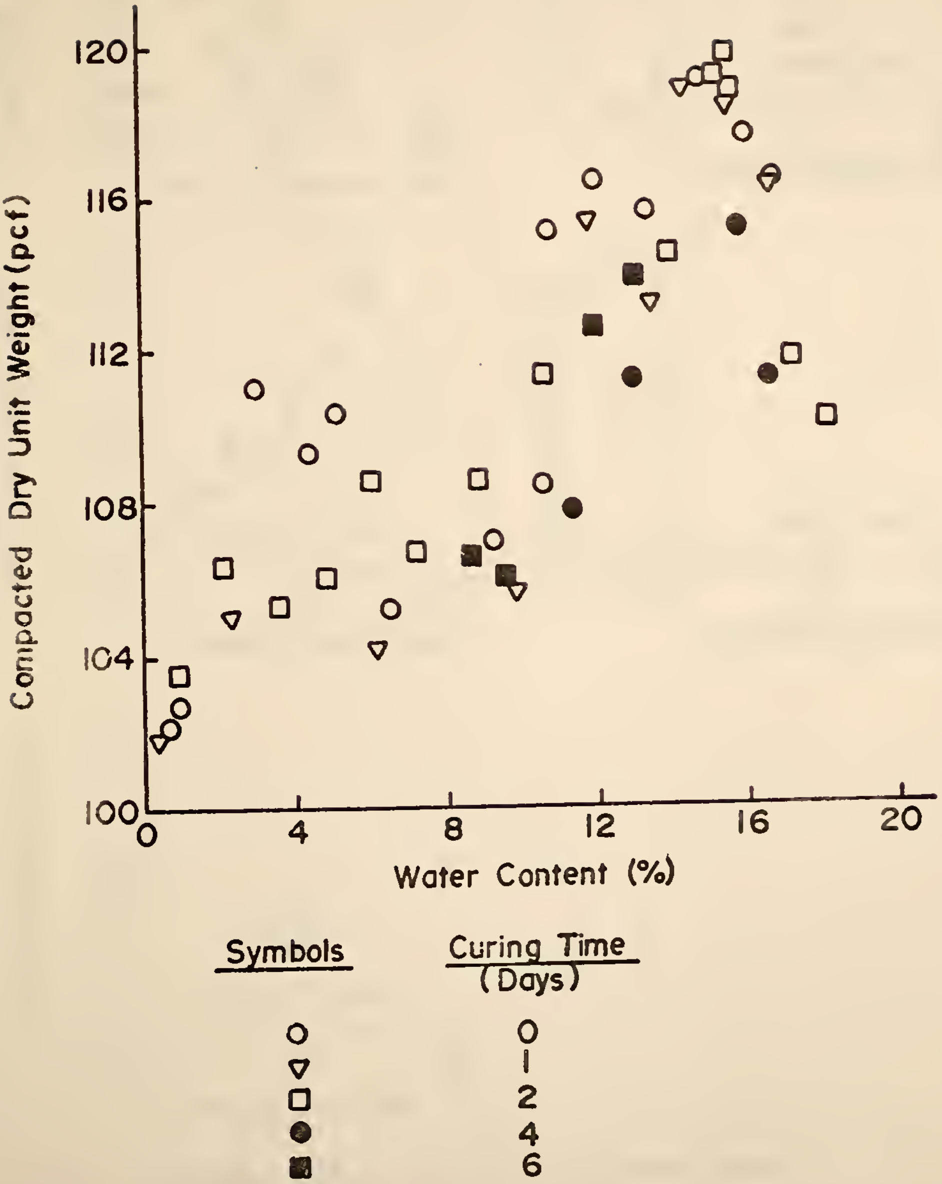
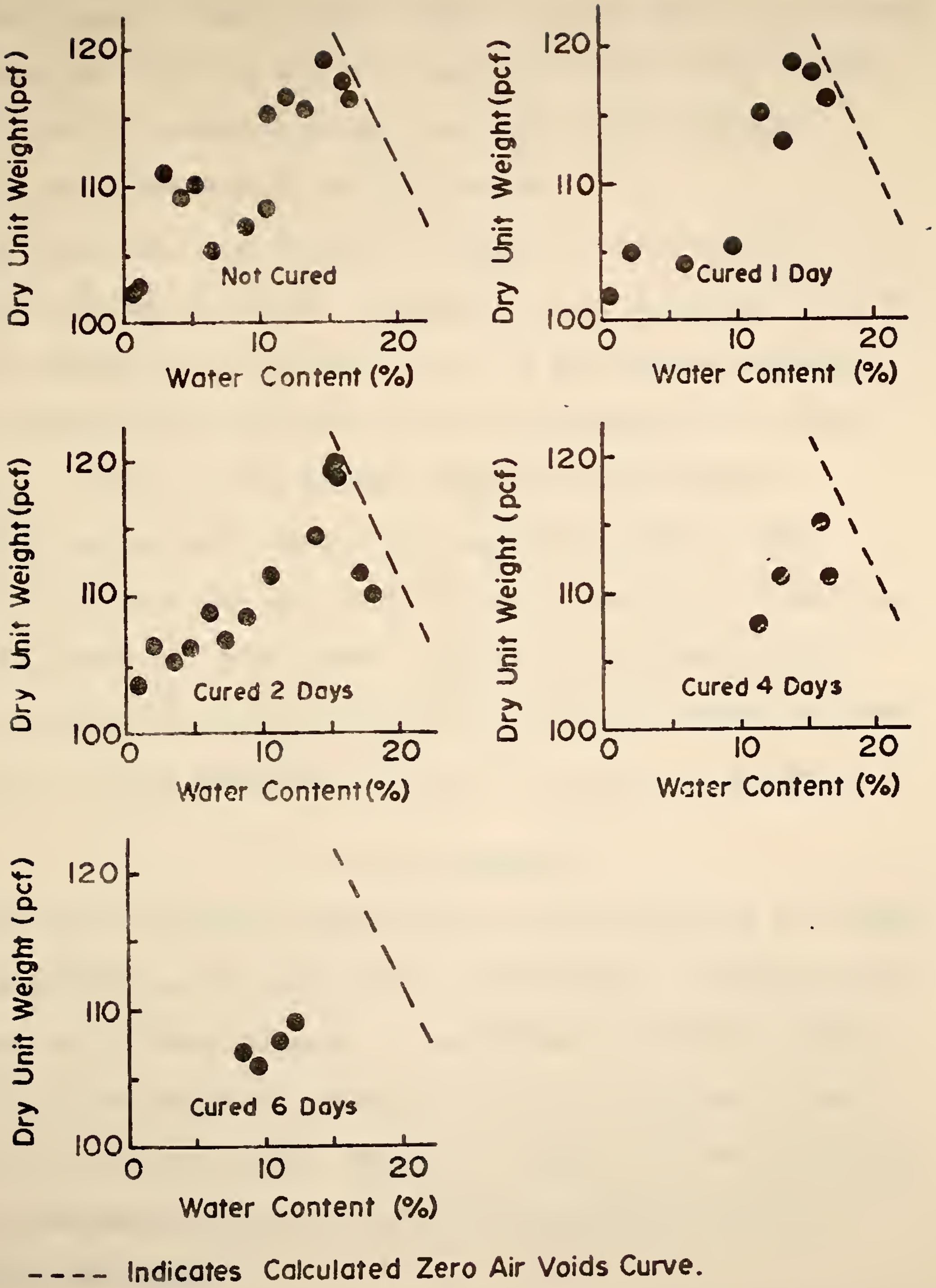


FIGURE 39 COMPARISON OF MOISTURE-DENSITY RESULTS FROM PROCTOR COMPACTION USING 2.75 LB DROP HAMMER FOR VARIOUS SAMPLE CURING TIMES.



---- Indicates Calculated Zero Air Voids Curve.

FIGURE 40 MOISTURE-DENSITY RESULTS FROM PROCTOR COMPACTION TESTS USING 2.75 LB DROP HAMMER FOR VARIOUS SAMPLE CURING TIMES.

pound drop Hammer. Double peaked compaction curves have been described by Wahls et al. (92) as a result of capillary forces in fine grained soils. There is no mention in the literature of this phenomena occurring in compacted shale samples however.

In Figure 40 it can be seen that there is little indication of a "dry-side" peak in the results from samples cured one or two days. The data from samples cured four and six days do not include compaction water contents in the same range, and so are not applicable to this discussion. Since it seems unlikely that capillary forces are significant in the shale samples which are predominantly coarse grained, the double peak must be due to some other effect of water on the shale particles. Such effects might include slaking, swelling and/or softening, however the investigation of these effects and their interactions during compaction is beyond the scope of this report.

4.4.4 Conclusions

Two types of impact compaction test were investigated to evaluate their suitability as an index test for characterizing the degradation of shales due to field compaction. Inital tests showed that neither full face impact compaction, modeled after the BSI Aggregate Impact Test, nor Proctor type impact compaction, adopted from the ASTM method D698-64T, enabled reproducable degradation measurements over an acceptable test range.

The Proctor compaction test was also used to investigate the effects of sample moisture and curing time on the compacted dry unit weights of shales. From the results obtained, no conclusions are possible regarding the effect of a curing period between the addition

of water and compaction testing.

A double peaked moisture-density curve was obtained from the results of some of the series of Proctor compaction tests. This phenomena appears to be due to some effect of water on the shale particles, but this effect was not specifically investigated.

Other sections of this report describe the effect of moisture on the strength and hardness of shale pieces, and the way in which moisture changes the particle gradation of shale samples over extended periods of time. It appears likely from the discussion above and in other sections of this report, that moisture has significant effects besides intergranular lubrication during compaction. An understanding of this phenomenon, based on further investigations, may significantly improve the state of the art of shale compaction, as applied to embankment design and construction.

4.5 Compaction-Degradation Classification

Results obtained from the four types of compaction test described in the preceding sections indicate that the static compaction method provided the most uniform relation between particle degradation, compaction effort and compacted unit weight. While there is no reported experience of this test being used to simulate actual field compaction, similar tests have been used in Great Britain and South Africa to evaluate the degradability of shales and other aggregate materials (11, 54, 76).

Accordingly, the static compaction test has been chosen as the most suitable method of comparing the compaction induced degradation characteristics of different shales. There are several factors which

were considered in making this choice.

1. The static compaction data was the most self-consistant over the effort range tested. The best relations between particle degradation, compaction effort and final unit weight, as measured by the second power of Pearson's correlation coefficient, were 0.90 for untamped static compaction, 0.88 for kneading compaction, and 0.73 for gyratory compaction. No significant relation could be determined for the results of impact compaction.
2. Static compaction tests are relatively convenient and inexpensive to perform. No specialized equipment, which is not available in conventional material testing laboratories, is required for performance of this test.
3. Static compaction tests can be performed quickly and easily. The nonhomogeneous nature of Indiana shales require not one, but several test repetitions to adequately describe any single parameter. The relative ease with which static compaction tests can be conducted makes it possible to perform several tests for any single investigation.

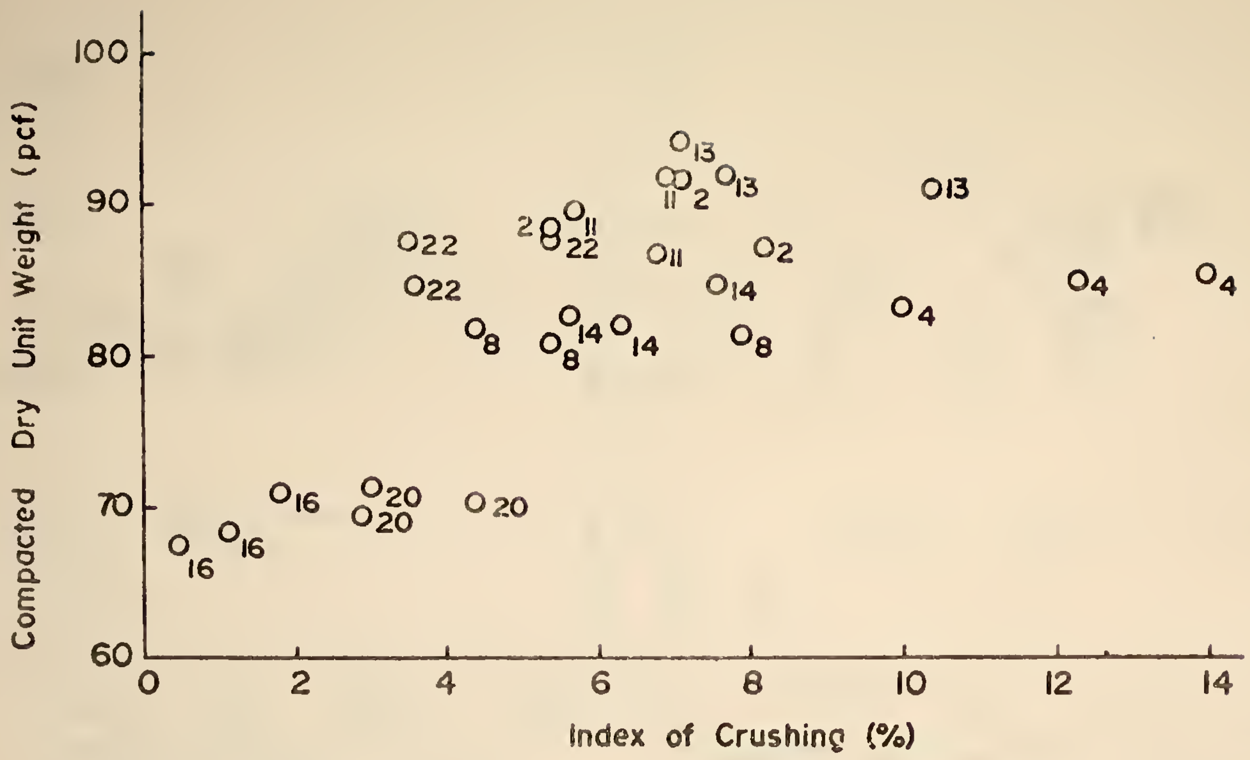
Furthermore it is hoped that the simplicity of this test will encourage other investigators to adopt it and produce a quantity of information describing all of the Indiana shales. When available, these data may be relatable to other shale properties, and amenable to analytical or empirical use in preparing field compaction specifications. Such specifications should include consideration of the compaction induced degradation, with respect to the stability of the completed

embankment.

One question remains before the static compaction test can be completely endorsed as an index or classification technique. To be of value in describing the degradability of various shales, an index test must be sensitive enough to distinguish between different shales, and also yield fairly consistent results for tests performed on any single type of shale. To evaluate this characteristic, static compaction tests were performed on three samples each of nine different shales.

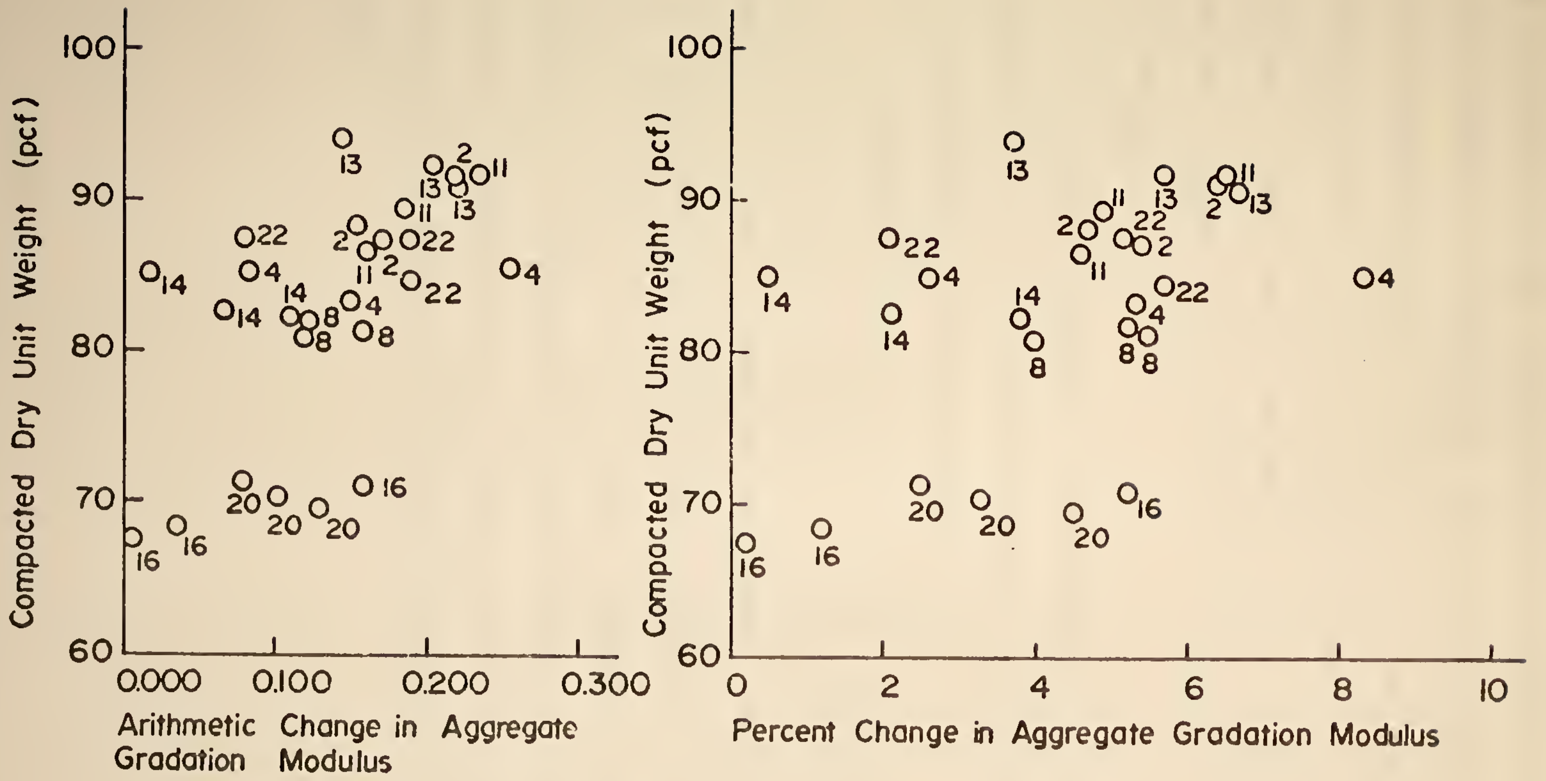
To compare the results of static compaction induced degradation, tests were performed with a loading pressure of 50 psi. Although tests could have been performed on a large consolidation test frame, or with a compressional testing machine and the loading scheme shown in Figure 28, the gyratory compaction apparatus was used for the sake of convenience. This choice was made because of the positive pressure control and built-in deformation gauge available on the gyratory loading apparatus. Furthermore only a limited amount of time remained between selection of this test method and completion of the total investigation. It was desirable to perform as many tests as possible without requiring the loss of time involved in scheduling access to a compression testing machine and technician. By using the gyratory testing machine, without the rotating arm mechanism, 27 static compaction tests were performed in less than eight hours.

The results of static compaction tests on the Attica model shale and eight other shales are shown in Figures 41 and 42. Data from these tests are tabulated in Appendix D.



Numbers Indicate Shale Type

FIGURE 41 DENSITY-DEGRADATION (IC) RESULTS FROM STATIC COMPACTION TESTS ON VARIOUS SHALES.



Numbers Indicate Shale Type

FIGURE 42 DENSITY-DEGRADATION (AGM) RESULTS FROM STATIC COMPACTION TESTS ON VARIOUS SHALES.

These test results show that static compaction can be used to discriminate between different types of shale. The dry unit weight values appear to segregate the shale tested into two groups: shales with compacted densities of about 70 pcf, and shales with compacted densities of 80 to 90 pcf. Although there is considerable scatter and overlap in the AA and PA degradation data, the index of crushing values indicate the same two groupings of shales as defined by the compacted unit weights.

Although there is significant variation in the IC data for the different shales (particularly Shale No. 4), this may indicate only the nonhomogeneity of the material tested.

In general it appears that the static compaction test can be used as a compaction-degradation index for evaluating the degradability of shales to be used in compacted embankments. Although initial results show fairly good discrimination and sensitivity characteristics, as well as a strong correlation between compaction effort, particle degradation and compacted unit weight, further research will be required to explore all of the facets of this test and to establish correlations between test data and the field behavior of compacted shales.

5. STRENGTH AND HARDNESS TESTS

5.1 Scleroscope Hardness Tests

5.1.1 Introduction

Several kinds of hardness tests are currently used in engineering practice to provide a measure of strength or resistance to yielding for various materials. In geological engineering, abrasion and dynamic hardness measurements (typified by the Mohr and Scleroscope tests, respectively) are most common (22).

Dynamic hardness tests provide a limited measure of several material parameters, primarily the elastic and plastic resistance to deformation. Hardness tests were considered applicable to this investigation because of the previously described need for a simple index test capable of providing a material strength paramter. In describing several forms of hardness tests, Deere and Miller report that although theoretical correlation is missing, empirical evidence relates the Scleroscope hardness number with rock properties otherwise measured as toughness, resilence, strength and elasticity (22).

A review of the literature indicates the earliest descriptions of the Scleroscope hardness test in geological engineering applications was the work described by Griffith (30) in 1937, and Obert et al (62) in 1946. Various investigators have used Scleroscope hardness tests as a measure of workability, toughness, unconfined strength, fabric variability, drillability, and abrasion resistance, for various rock

materials (22, 48, 62, 75). Statistical correlations have been developed to indicate those properties and their variability with respect to Scleroscope hardness, and are reported in the literature previously cited.

Prior to this investigation there was no recorded evidence of the use of the Scleroscope hardness test to discriminate among various Indiana shales and their engineering properties. In considering applications to improving the state of the art of shale embankment construction, the Scleroscope test was considered for the following reasons.

Hardness tests have been used to indicate the strength of various rock materials. The parameter of shale strength is of interest in anticipating problems in the excavation and placement stages of embankment construction. The strength or hardness of shale particles is also a major factor in determining the extent of degradation during embankment compaction. Finally, previous work described by Deere and Miller (22) indicated that Scleroscope tests were easily performed on rock specimens, with a high degree of repeatability; and that of all the dynamic hardness tests, the Scleroscope test was most suitable for rock study.

On this basis a testing program was initiated on shale samples provided by the ISHC.

5.1.2 Test Description and Sample Preparation

The Scleroscope hardness test is a dynamic rebound test. Performance of the test consists of dropping a diamond tipped steel rod onto the surface of the test specimen from a fixed height, and

measuring the maximum height of rebound of the rod. The operation of elevating and releasing the rod is an automatic sequence which accompanies the manual rotation of a control knob on the Scleroscope apparatus. This same sequence automatically fixes the maximum rebound height of the rod after it strikes the specimen. Counter-rotation of the control knob causes this rebound height to be recorded on a dial gauge (77). The dial gauge readings are in Scleroscope hardness units, which range from 0 to 120. A Shore Recording Scleroscope, Model D, was used in this investigation.

Following a suggested standard test procedure, several hardness measurements were made on each shale specimen and an average hardness was reported for each test series (77). According to the manufacturer, The Shore Scleroscope is accurate for specimens with a minimum thickness of 0.005 inch to 0.015 inch, depending on the material properties (77). This indicates that the hardness of individual lamina in a single shale specimen could be measured and contrasted. The averaging process tends to give a gross hardness measurement however. Initial Scleroscope hardness tests on shale pieces indicated the following test constraints.

1. The measured hardness was very sensitive to small discontinuities, i.e., "roughness", on the specimen surface being tested.

2. The measured hardness for single samples tended to increase as the samples dried out.

These observations led to the development of a standard sample preparation and test procedure, as described below.

Shale specimens were selected for Scleroscope testing on a random basis from material which was on hand, for tests on Shale Nos. 2, 4, 8, 11, 13, 14, 16 and 20. The selected specimens were all about two inches square by one inch thick, with thickness being measured perpendicularly to the bedding planes. The samples were roughly squared by sawing. A steel band saw was used for cutting most of the specimens, however the vibration from the saw motor caused some of the softer shales to fall apart during this process. The shales which could not stand up to the band saw were cut by hand, using a carpenter's coping saw.

Following the rough saw cutting, the test surfaces were smoothed with a wood rasp and/or coarse quartz sandpaper. Specimens were tested on a single lamina per side, that is, the hardness was only measured perpendicularly to the bedding planes. Final sample preparation consisted of continued sanding with successively finer grades of quartz and emery paper. The sample preparation was considered complete when the shale surface to be tested was polished smooth with no flaws or discontinuities visible or discernable by touch. The sample was then weighed for future water content determinations, and tested.

Each shale surface was tested from nine to twenty-four times, and an average hardness was calculated from the results of these tests. The Scleroscope was recalibrated between every second sample tested on any given day, and at the beginning and end of each day's testing. The average hardness calculated for each shale specimen was corrected, if required, by these calibration checks.

Following testing, the shale specimens were placed in an oven set

at 40°C. Tests were repeated at intervals of 24 and 48 hours, after which the shale specimens were placed in another oven set at 105°C, and tested after a further 24 hours. Prior to each test series, the shale specimens were removed from the oven and allowed to cool in a desiccator jar to prevent moisture gain. When cool, the specimens were weighed and then tested. In this way, each specimen was tested a number of times over a range in water contents. Final tests were made after the specimens had reached a constant weight in an oven set at 105°C. This drying process took as long as 45 days in some cases.

The test procedure was altered somewhat for Shales Nos. 1, 3, 6, 7, 9, 10, 17, 18 and 21. Only one or two specimens of each of these shales were available, and no attempt had been made to store them at their in situ moisture content. A reduced drying range was thus used for these shales.

5.1.3 Test Results

In all, more than 5000 single hardness measurements were made on samples from seventeen Indiana shale formations. The averaged test results, corrected for instrument drift where necessary, are tabulated for the various shales at various moisture contents, in Appendix E.

Before discussion of specific test results, some comments should be made on the test procedure adopted and observations made during the tests.

First of all, the sample preparation procedure was extremely time consuming and may have been too restrictive. Early tests showed that the scatter in hardness data for a single specimen surface was considerably reduced when that surface had been carefully polished.

However the cutting and sanding process usually required about three hours per specimen. During this time several of the shale specimens broke apart, usually along the bedding plane orientation, and new specimens had to be prepared. This suggests the possibility that the test results reported herein are biased in favor of the stronger, more durable shales, which were able to survive the preparation procedure.

Additionally, several of the specimens fell apart during the oven drying periods, or in handling when removed from the ovens. This reinforces the bias mentioned above.

Several of the shales developed small hairline cracks and/or exhibited chipping and flaking around the edges during oven drying. As the samples were dried in small aluminum pans, no weight loss occurred to invalidate the moisture content observations, however there was a marked drop in hardness around the deteriorated area. Laboratory personnel were instructed not to test the areas of the samples which showed such deterioration, and this may have further biased the test data.

Even with these restrictions, the Scleroscope hardness data seem reasonably consistent except at the zero moisture content condition. Specific test results show that the variability in measured hardness increased as the shale specimens became drier.

It appears that the hardness measurements reflect changes in the shale microstructure which accompany moisture loss. The hairline cracks and chipping which were observed can be considered as macroscopic evidence of deterioration of the shale, in contrast to the increase in hardness which accompanied drying.

Figure 43 shows the change in hardness with drying for all of the shales tested. As Figure 43 indicates, there is a general increase in shale hardness at lower levels of moisture content. The deterioration of the shale, especially at zero or near zero moisture content, appears to be responsible for the increased data scatter in this range.

The tendency for lower hardness values at higher levels of moisture content is also clearly evident in the test results for each of the shales considered individually. An example of this is shown in Figure 44, which presents the Scleroscope test results for Shale No. 11, the Attica model shale. Figure 44 also shows the high data scatter at or near $w = 0$.

Multiple regression analyses were performed to determine the correlation between Scleroscope hardness, H, and moisture content, w, for the various shales. The results of these analyses are shown in Table 12. Pearson's R^2 correlation coefficient is also shown, as a measure of the validity of the correlations (60). No significant relation between hardness and moisture content was found for the tests on Shale Nos. 6, 9, 10, 13, 17, 18 and 21.

Significant correlations were found for Shale Nos. 1, 2, 3, 4, 7, 8, 11, 14, 16, and 20. For these ten shales, the hardness test results all show the same general pattern for changes in moisture content. All of the shales showed a proportional decrease in hardness of one to four times the increase in moisture content. The average slope of the Scleroscope hardness vs. moisture plots is about - 2.4.

Examination of the calculated values of Scleroscope hardness from the hardness vs. moisture relations indicate the shales tested fall

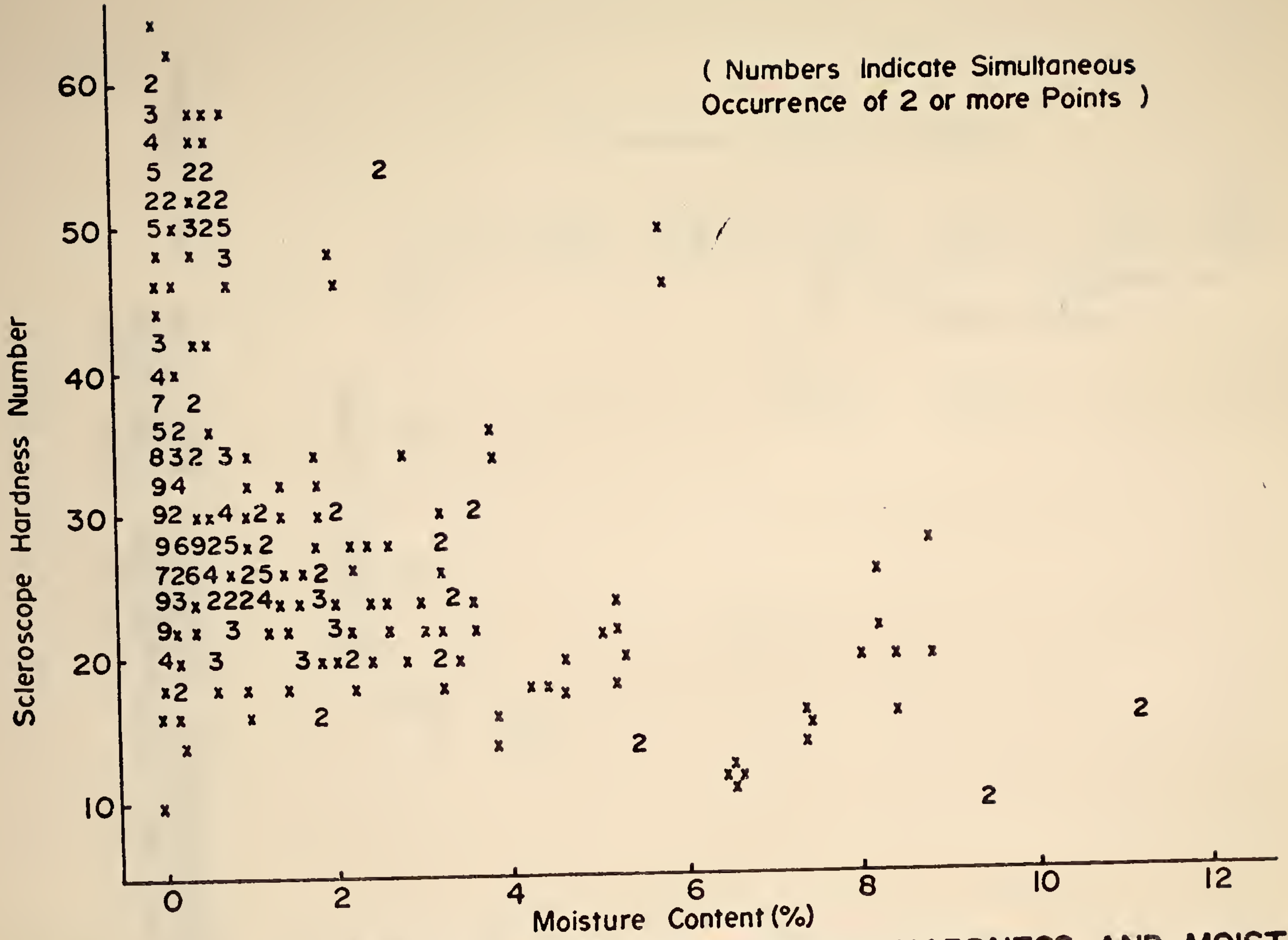


FIGURE 43 RELATION BETWEEN SCLEROSCOPE HARDNESS AND MOISTURE CONTENT FOR ALL SHALES TESTED.

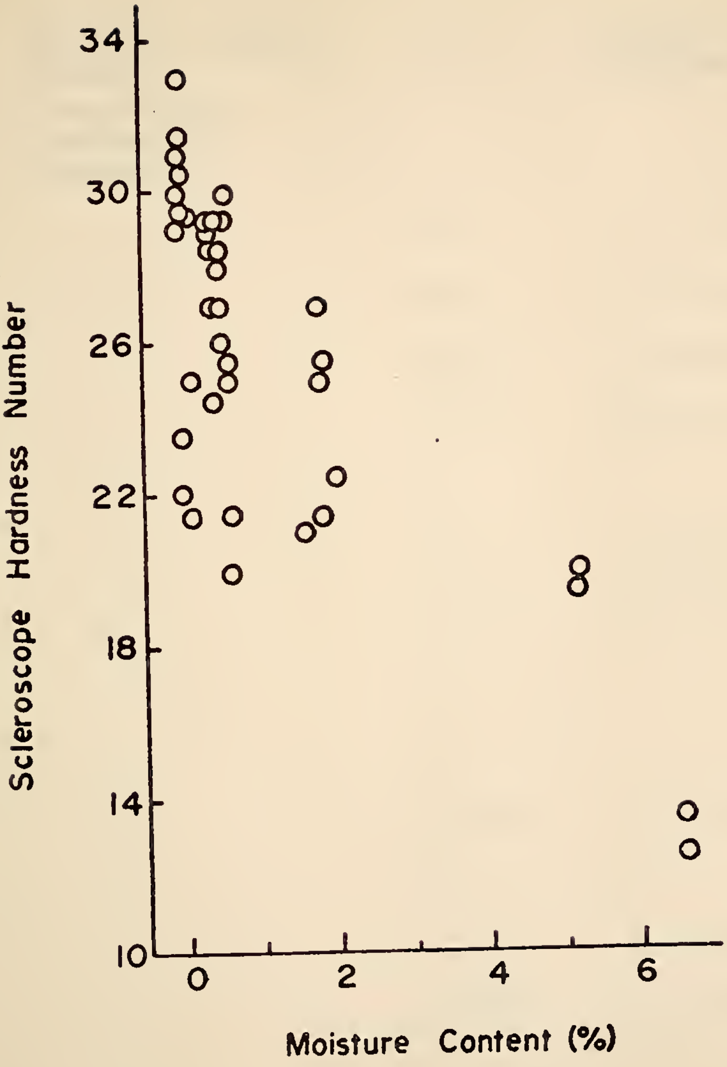


FIGURE 44 RELATION BETWEEN SCLEROSCOPE HARDNESS AND MOISTURE CONTENT FOR SHALE NO. 11.

Table 12.

Results of Regression Analyses Relating Scleroscope Hardness With Moisture Content For Various Shales.

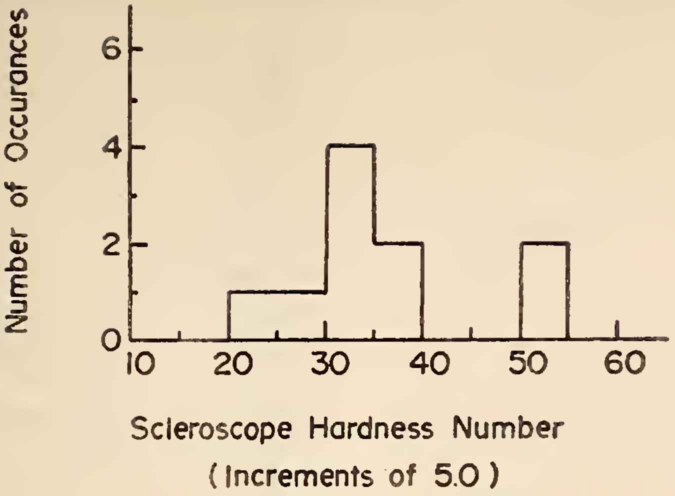
Shale Number	Number of Averaged Hardness Observations Included In Regression	R^2	Derived Correlation Equation
1	8	0.76	$H = 37.3 - 1.9(w)$
2	28	0.52	$H = 31.3 - 2.8(w)$
3	4	1.00	$H = 39.8 - 3.0(w)$
4	63	0.40	$H = 31.4 - 3.0(w)$
6	4	*	
7	8	0.91	$H = 51.6 - 2.0(w)$
8	29	0.26	$H = 32.2 - 1.3(w)$
9	8	*	
10	9	*	
11	48	0.61	$H = 28.3 - 2.2(w)$
13	18	*	
14	58	0.43	$H = 32.1 - 2.4(w)$
16	40	0.35	$H = 53.8 - 4.0(w)$
17	4	*	
18	4	*	
20	38	0.62	$H = 21.4 - 1.0(w)$
21	11	*	

* Indicates Insignificant Correlation At The 99% Probability Level

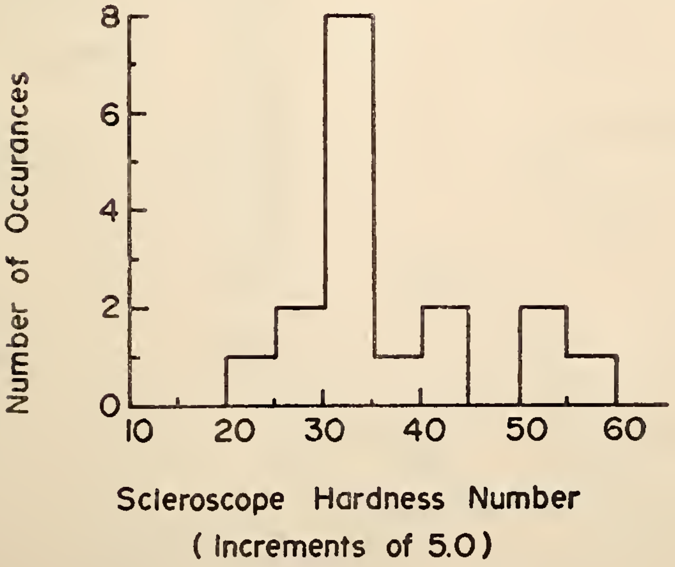
into two general hardness groups. The first group, with an average oven-dry hardness of about 32, is typified by Shale Nos. 2, 4, 8 and 14. The second group, with an average oven-dry hardness of about 52, consists of Shale Nos. 7 and 16. The discrimination between these two groups of shales is more clearly seen in Figure 45, which shows a frequency distribution plot of the test results segregated in hardness number increments of 5.0.

Figure 45a shows the distribution of average hardness computed from the equations presented in Table 12. Figure 45b shows the distribution of the average values of hardness actually measured for each of the shales tested. In both cases a segregation of "hard" and "intermediate" shales is evident. Such a segregation may be a useful tool for classifying shales on the basis of hardness. More data could tend to "fill out" the frequency distributions shown in Figure 45 however, leading to a normal or Gaussion distribution for the data (59).

Figure 46 shows the mean values (used in Figure 45b) and ranges of the hardness data measured in all of the Scleroscope tests on oven dried samples. The data in Figure 46 show considerable overlap and large scatter for some of the shales. This tendency suggests that the Scleroscope hardness test may not be of much value as a classification index unless a large enough quantity of data is available to allow statistical rejection of extreme values. The validity of a hardness index or of a correlation between Scleroscope hardness and other engineering properties of Indiana shales cannot be established without further testing and analysis.



a) Calculated Results of Significant Correlations



b) Actual Test Results

FIGURE 45 FREQUENCY DISTRIBUTION BAR GRAPHS FOR SCLEROSCOPE TEST ON OVEN DRIED SAMPLES OF VARIOUS SHALES.

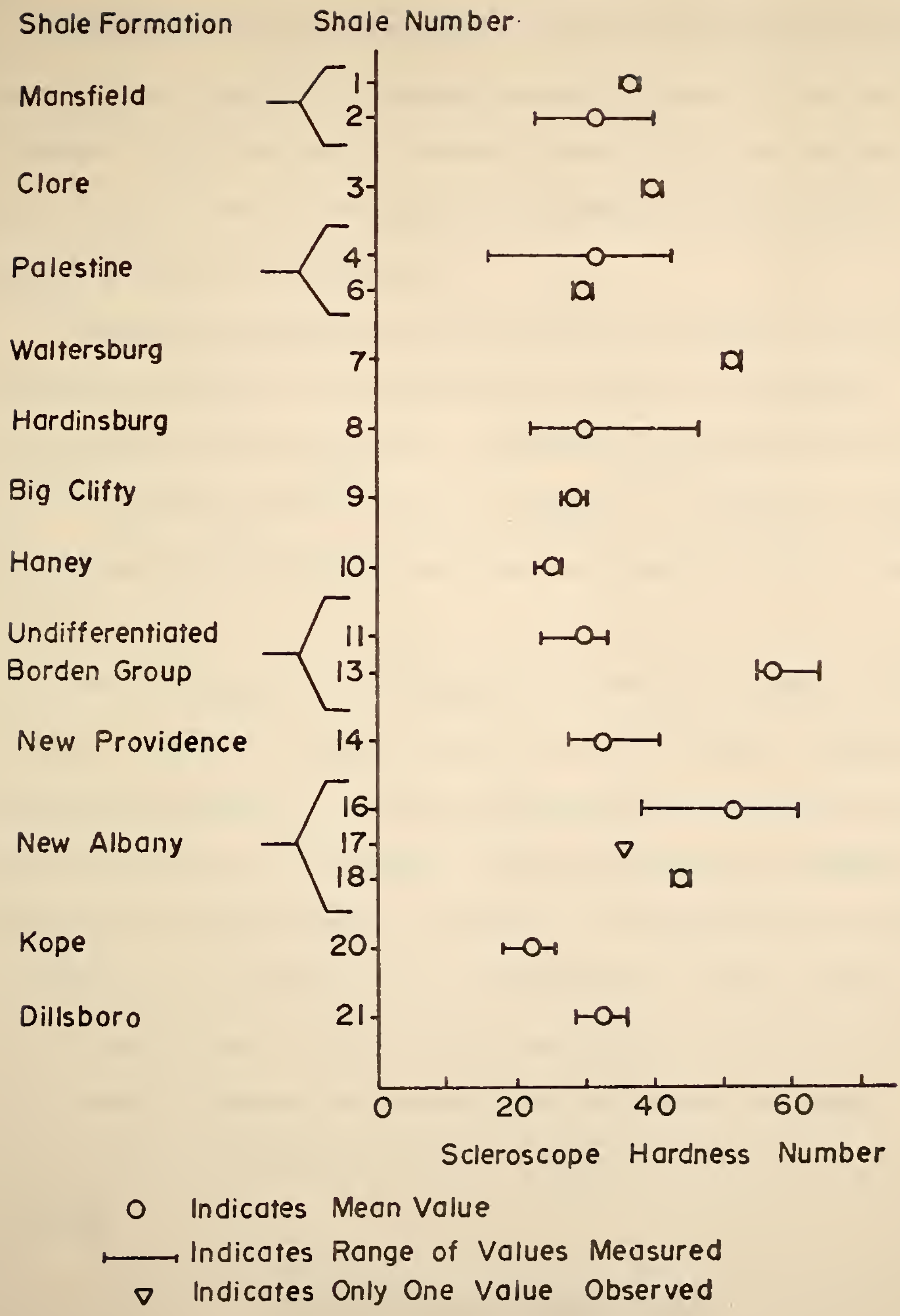


FIGURE 46 AVERAGE VALUES AND RANGE OF SCLEROSCOPE HARDNESS FOR VARIOUS SHALES AT ZERO MOISTURE CONTENT.

5.1.4 Conclusions

Results of Scleroscope hardness tests on seventeen Indiana shales show a significant trend towards greater hardness at lower moisture contents. This tendency is somewhat obscured by data scatter, which may be due to the obvious sample deterioration that occurred as the shale specimens were dried out.

Statistical regression analyses showed a similar hardness vs. moisture relation for the ten shales for which a significant correlation could be obtained. Frequency distribution of the average oven dry hardness values from the correlation equations for these ten cases suggest that Indiana shales may be classified into either of two groups.

The two groups are distinguished by the difference in their average Scleroscope hardness numbers, which are about 32 and 52. Frequency analysis of the actual average values of measured oven dry hardness confirm the existence of these two groups. A classification index may be established on this basis. The validity of such an index, as well as its relation to other properties of Indiana shales has not been demonstrated. Further testing and analysis may be of value in this respect; however the difficulties encountered in sample preparation, and the data scatter, indicate this will be a difficult problem.

5.2 Point Load Strength Test

5.2.1 Introduction and Background of Test

Current rock mechanics and materials testing literature describes

several kinds of rock strength tests (19, 20, 28, 45, 61). Unfortunately all of these tests have one or more disadvantages that have prevented universal acceptance of any single technique.

Franklin (28) noted that uniaxial and triaxial compression tests are most commonly used for determining the shear strength of rock materials. While such tests are fairly common for design purposes, they are not suitable for classification tests because of the special preparation needed to cut and smooth samples into standardized dimensions (2:1 cylinders or prisms). Furthermore, results of these tests are biased in favor of the stronger samples which are able to survive the preparation procedure (28).

Most of the Indiana shales used in this research are not suitable for unconfined or triaxial testing on intact pieces. The diamond drilling and cutting equipment which was available used water to cool the tool face and remove cuttings; this water also caused slaking of the sample. Experience on this project also demonstrated the impossibility of maintaining an original moisture content during the time required for lapping the ends of each sample.

Other tests present disadvantages in terms of the quantity or quality of sample material required. Since initial field investigations frequently consist of diamond drill coring or hand collected bag samples, tests which require large amounts of material may be impractical to perform (38). Standard uniaxial compression tests require only small samples, but each sample must be free from gross defects such as cleats, healed fractures, or intrusive veins. One recent project (6) included two 25 foot core borings from which

only 20 feet of core material could be recovered. Of this, only one piece of the core was large enough to be recut and tested in uniaxial compression. Such experience demonstrates the need for less exacting specimen requirements for classification tests.

In order to avoid the problems of special sample preparation, "irregular lump" strength tests have been devised. Hobbs (35) and Franklin (28) have described developmental work by Protodyakonov. This work led the International Bureau of Rock Mechanics to recommend a standard test procedure in 1961. The first indirect tensile strength test was reported by Carniero and Barellos in 1953, the so-called "Brazilian Test". The combined influence of these two test methods led to the development of the point load (indirect tensile) strength test (28, 68). This test has several attributes which will be described in detail.

The point load strength test (PLS test) does not require any particular sample preparation. Specimens as small as 10 cc. (about 30 grams) were tested quite satisfactorily, as were much larger pieces, during this investigation. The upper size limit of sample pieces is governed by the capacity of the loading apparatus. Some of the tests reported herein were conducted on shale pieces whose smallest dimension was in excess of two inches across.

Furthermore, test results do not seem dependent on the sample shape. While various theoretical analyses of indirect tensile failures have been developed for differently shaped loaded areas (27, 28, 34, 45), the differences are small in relation to the effects of nonhomogeneity, anistropy, and non-elastic behavior in the rocks

tested. Using photoelastic techniques, Hiramatsu and Oka (34) observed the same stress fringe patterns for several differently shaped bodies stressed under point loads. Nevertheless, the PLS test method proposed by the International Society for Rock Mechanics (43) recommends testing only samples with a longest diameter no more than 40% greater than the smallest diameter. In most cases this criterion was used in this investigation. Tests performed by Jaeger (44) showed similar stress distribution regardless of sample shape, and Franklin (28) has noted that protrusions remote from the sample's loaded diameter have little effect on strength.

Tensile failures occur in PLS test samples as a result of stresses developed at right angles to the loaded diameter of the specimen. Actually the sample yields and crushes at the load platen contacts, so the "point" load is in reality applied over some finite area. This is not significant (except from a theoretical point of view, see Hobbs, reference 36) for two reasons.

a. The sample's resistance to crushing at the points of applied load is a material property in the same sense as the tensile strength. If the same size platens are always used, the effect of crushing is constant for each material type.

b. The critical zone of stress for failure initiation is in the interior of the sample, hypothetically at the midpoint of the loaded diameter. Therefore the surface condition of the sample has little effect on the measured strength.

The concept of the loaded diameter as the zone of highest stress within the sample also eliminates the need for concern about whether

imperfect orientation of the platens will cause load eccentricities, resulting in undesirable stress concentrations. However, imperfect alignment of the platens may change the orientation of the failure surface (with respect to cleavage or bedding planes for example) within the sample. Tests reported herein were on samples loaded perpendicularly to the bedding planes.

Finally, the relatively short time required for each PLS test, less than 10 minutes generally, enables preservation of the desired specimen moisture content. In summing up the advantages of PLS tests, Franklin (28) has commented, "...time that would normally be taken in preparation should be utilized to allow further testing". This is important because the wide variability in shale materials frequently necessitates statistical interpretation of data. Increasing the number of strength tests will improve such interpretations.

Other indirect measures of rock strength have been attempted. Chapman (16) has described the use of the Schmidt Rebound Hammer for testing Indiana shales, and D'Andrea et al (20) have attempted to predict the strength of rocks using a variety of other properties. Both of these studies have reported their own particular problems.

5.2.2 Description of Point Loading Apparatus and Test Procedure

The PLS test apparatus used in this study is shown diagrammatically in Figure 47. The base of the device rigidly supports the lower platen and the two guide bars. The upper platen is mounted at the end of the loading ram which, with the guide plate, slides freely on the guide bars. A dial gauge, reading to the nearest one thousandth of an inch, measures the height of the guide plate above the base plate.

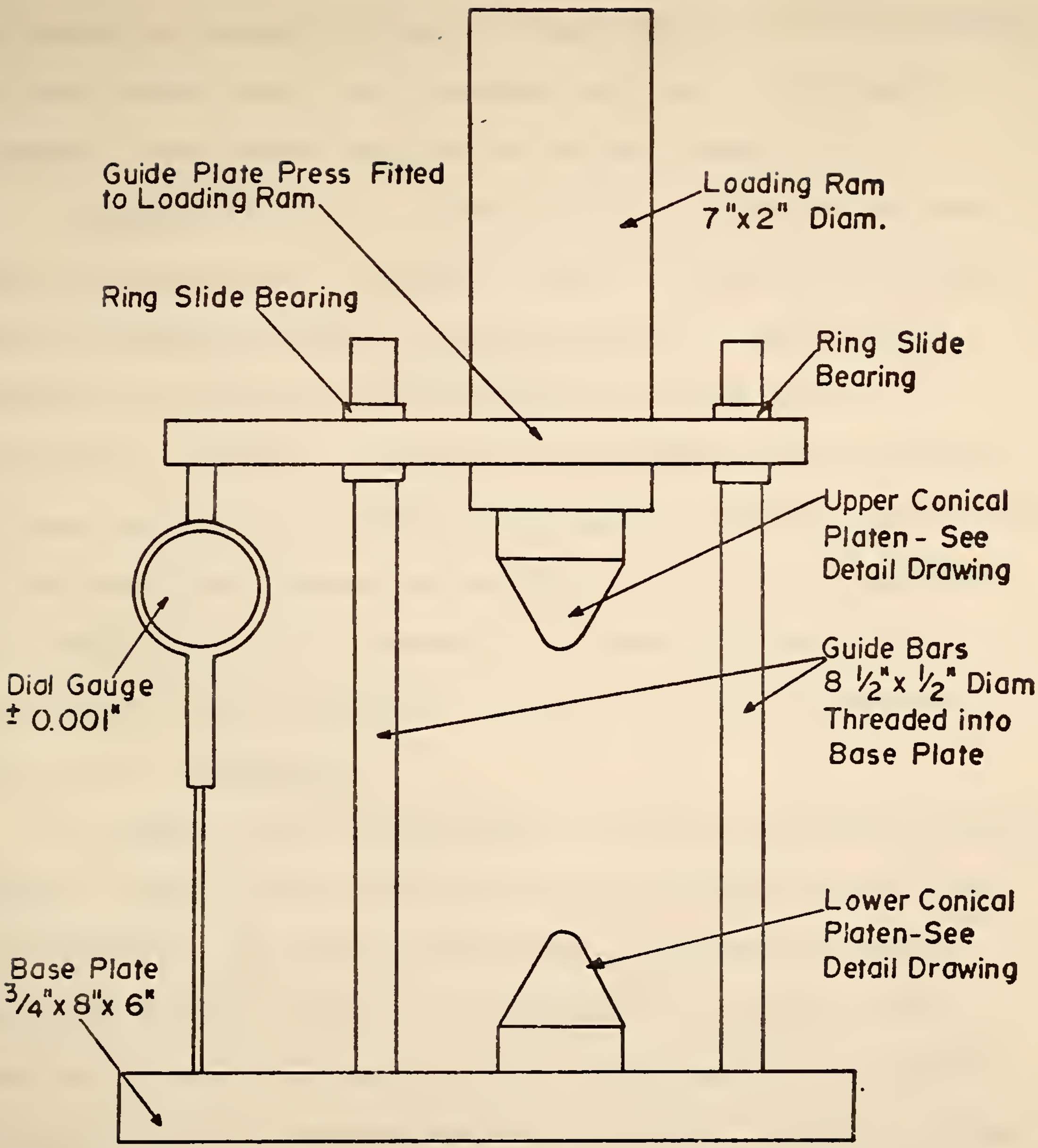

Guide Plate Press Fitted to Loading Ram
Loading Ram 7"x2" Diam.
Ring Slide Bearing
Ring Slide Bearing
Upper Conical Platen - See Detail Drawing
Dial Gauge ± 0.001"
Guide Bars 8 ½"x ½" Diam Threaded into Base Plate
Lower Conical Platen-See Detail Drawing
Base Plate ¾"x 8"x 6"

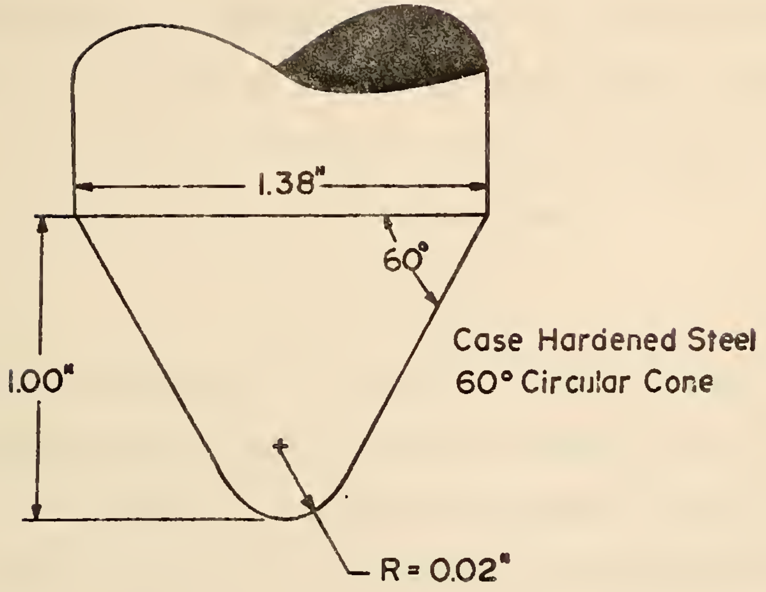
FIGURE 47 SIDE VIEW OF POINT LOAD TEST APPARATUS.

Figure 48 shows the shape and dimensions of the loading platens. The platens are sperically tipped circular cones of case hardened steel. The platen design used on this apparatus conforms to the proposed standards of the International Society for Rock Mechanics (43).

To conduct a PLS test, a piece of shale is placed between the upper and lower platens. The weight of the upper platen and load ram holds the specimen in place. Although samples were approximately centered on the lower platen, no particular care was required to maintain their placement. Initial load application caused some slight rotational adjustment so that the compressional load transmitted by the platens always acted through the shortest dimension of the specimen cross section. An initial reading of the gauge indicates the guide plate height, and by calculation, the sample thickness between the platen tips is determined.

The loading force acting through the platens compresses the shale specimen, causing tension stresses to develop perpendicularly to the loaded diameter. The load was applied by an electrically driven compression testing machine, at a constant rate of strain of 0.01 inches per minute. The load resistance of the sample was monitored through a 5000 pound capacity, SR4 type, load cell. Stress and strain were monitored continuously with a dual channel strip chart recorder.

Several methods for calculating the tensile stress at failure have been proposed for PLS tests (20, 34, 44, 57). These analyses (theoretical and empirical) all include a term equal to the ratio of ultimate compression load and the square of the original sample thickness. Various coefficients, most commonly based on specimen



Upper Platen Threaded into Axial Ram.

Lower Platen Threaded into Base Plate.

FIGURE 48 DIMENSIONS OF PLATEN FOR POINT LOAD TEST APPARATUS.

geometry, have been proposed to "correct" this term. Franklin (28) and Jaeger (44) review and compare some of these analytical treatments of the tensile stress. Poulos and Davis (65) describe the stress field resulting from an ideal point load. Roark (71) presents stress equations for the case of a spherical load platen, similar to the spherically tipped conical platen actually used.

Following the conclusions of Franklin (28), PLS results are reported herein as the ratio of maximum compressional load and the square of the initial platen separation (initial sample thickness). No correction factor is included in these data.

5.2.3 Test Results

Results of PLS tests in this investigation have been considered in two categories because of the effects of specimen moisture. PLS tests on oven dried specimens at "zero moisture content" ideally provide a reference test from which to compare directly the properties of different shales. PLS tests on specimens of the same shale at various moisture contents show a range in strength which would confuse any comparison among different shale types. However this variation may be significant in evaluating the degradation potential of any particular shale.

The first series of PLS tests was performed on Scleroscope test specimens which had been oven dried to a constant weight. Limited quantities of six shales permitted only one PLS test on each. The results of these tests are tabulated in Appendix E. The average results and range of data for all PLS tests at zero moisture content are shown in Figure 49. The vertical axis of Figure 49 shows eighteen

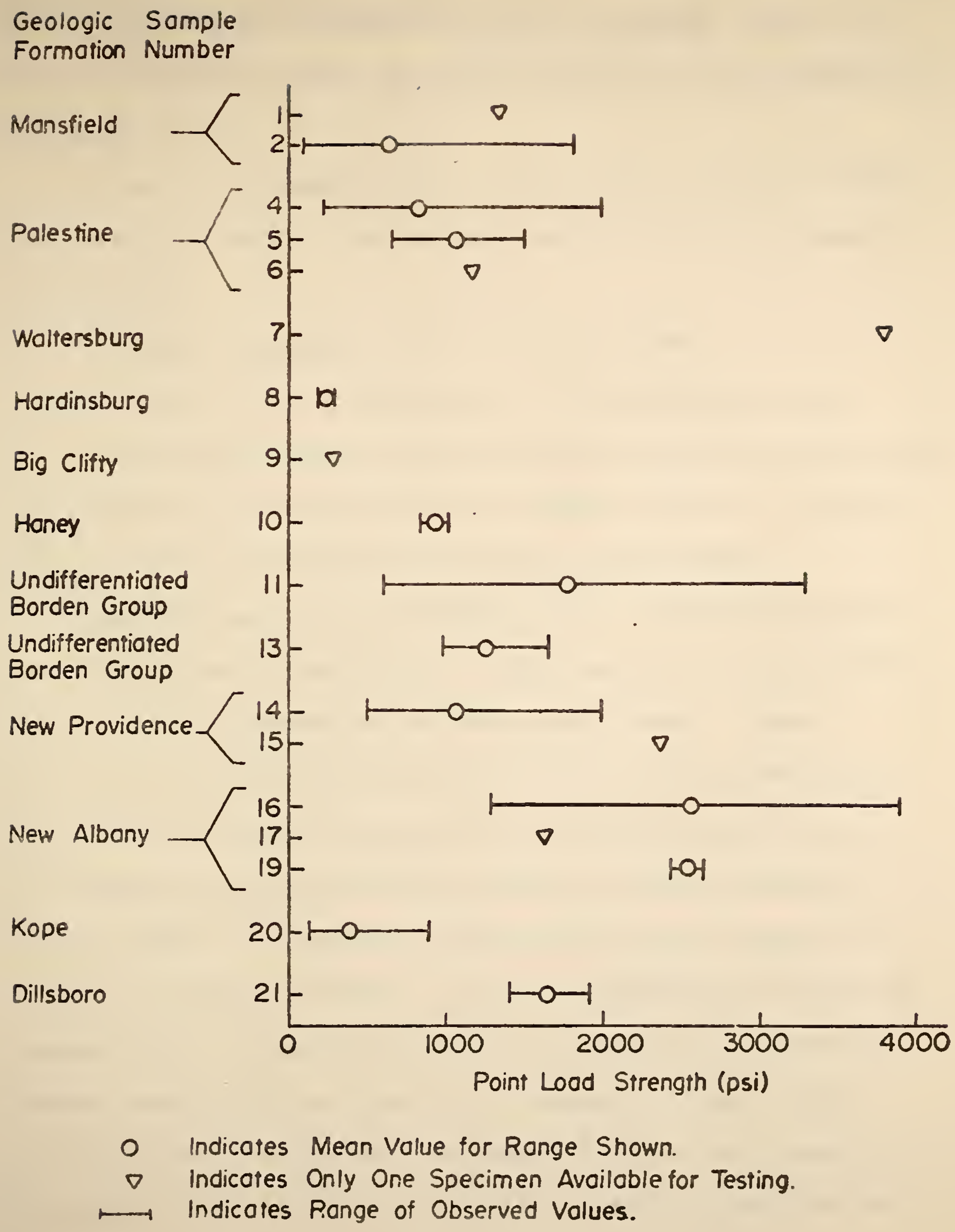


FIGURE 49 COMPARISON OF PLS VALUES FOR 19 SHALES AT ZERO MOISTURE CONTENT.

shale types arranged in order of geological occurence. There is no obvious correlation between the formation and point load strength for the shales tested.

Several of the shales tested in an oven-dried condition showed a wide range in strength. This variation may be due to one or more of the following:

a. natural inhomogeneities in the test specimens,

b. effects of size and/or shape of the test specimens, and

c. non-uniform changes in the shale due to the oven-drying process.

Based on the discussion in the preceding section, it seems that item b. is the least prominent cause of the data scatter.

A gradual oven drying procedure was used to remove the moisture from each specimen before testing. This procedure is described fully in the section on preparation of Scleroscope samples. Visual examination of smooth sanded surfaces on the test specimens showed gradual deterioration of the shale as moisture was removed. This deterioration appeared as flaking, chipping, and the formation of hairline cracks. The extent of deterioration was not constant for all of the specimens observed. No way of observing the formation of interior cracks or other deterioration was possible as the drying process extended into the interior of a specimen. Since the PLS tensile failure is initiated in the interior of the test specimen, the likelihood of internal flaws, similar to drying induced surface flaws, must be considered in explaining the data scatter.

Nine shales were subjected to PLS tests at a variety of moisture contents. The results of these tests are tabulated in Appendix E.

Experience gained from the long-term soaking degradation tests indicated the impracticality of artificially inducing high moisture contents in the shales. The prospect of inducing internal flaws through oven drying also prohibited artificially reducing the moisture contents. Thus only shales which showed some variation in water content in the storage containers are included in this category. The shales tested had been stored for 12 to 36 months under "equilibrium" conditions, and this was considered suitable for comparing the change in strength with moisture content. A complete description of sample storage procedures and the relative moisture variations are included in Appendix A. To reduce the drying time required in measuring water content, the test specimens were broken up with a hammer following PLS testing. This enabled relatively fast (2 to 5 days, as opposed to 30 to 40 days) oven drying, and insured that the water content of each specimen could be accurately determined.

Of the nine shales tested at various moisture contents, four showed recognizable linear relations between water content and point load strength. These are Shale Nos. 2, 4, 11, and 20. With the exception of one very anomalous value, the PLS data for Shale No. 16 also shows a linear relation with moisture content. For these five shales, there is a clear trend of decreasing strength at higher moisture contents. The PLS test data for Shale Nos. 8, 13, 14, and 22 showed too much scatter or too small a moisture content range for any trend to be recognized.

Despite the fact that not all of the shale types individually showed lower strength values at higher moisture contents, this trend

is very evident when all of the shale samples are compared together. Figure 50 shows the results of 190 PLS tests on the shale samples previously mentioned. The data points to the left of the vertical dashed line represent oven dried specimens, the others represent the various storage moisture conditions. It appears that oven drying tends to both (a) increase the shale strength by reducing the moisture content, and (b) decrease the strength through alteration of the material structure. This alteration has already been described as the formation of hairline cracks and other deterioration. The relation between strength reduction due to artificial drying, and the rate of drying, was not investigated.

The change in shale strength with change in moisture content has been previously noted by Bieniawski (9) and others (7, 63). For compaction of shale, an increase in moisture content on the surface of pieces was reported to improve breakdown on one project (84).

Simple regression analyses were performed on the test data for the four shales where a linear relation between strength and moisture content seemed evident. Computerized linear regressions enabled the determination of a "best fit" through the plotted data for each shale type. The second power of Pearson's correlation coefficient, R^2, was used to evaluate the accuracy of this line in describing the observed data (60). Because of the range in PLS values for the zero moisture condition, a question arose regarding how to treat these data. Regressions were performed using the non-zero moisture content test data ($w > 0$) test data, all of the test data ($w \geq 0$), and the non-zero moisture content test data with the mean value of the zero moisture

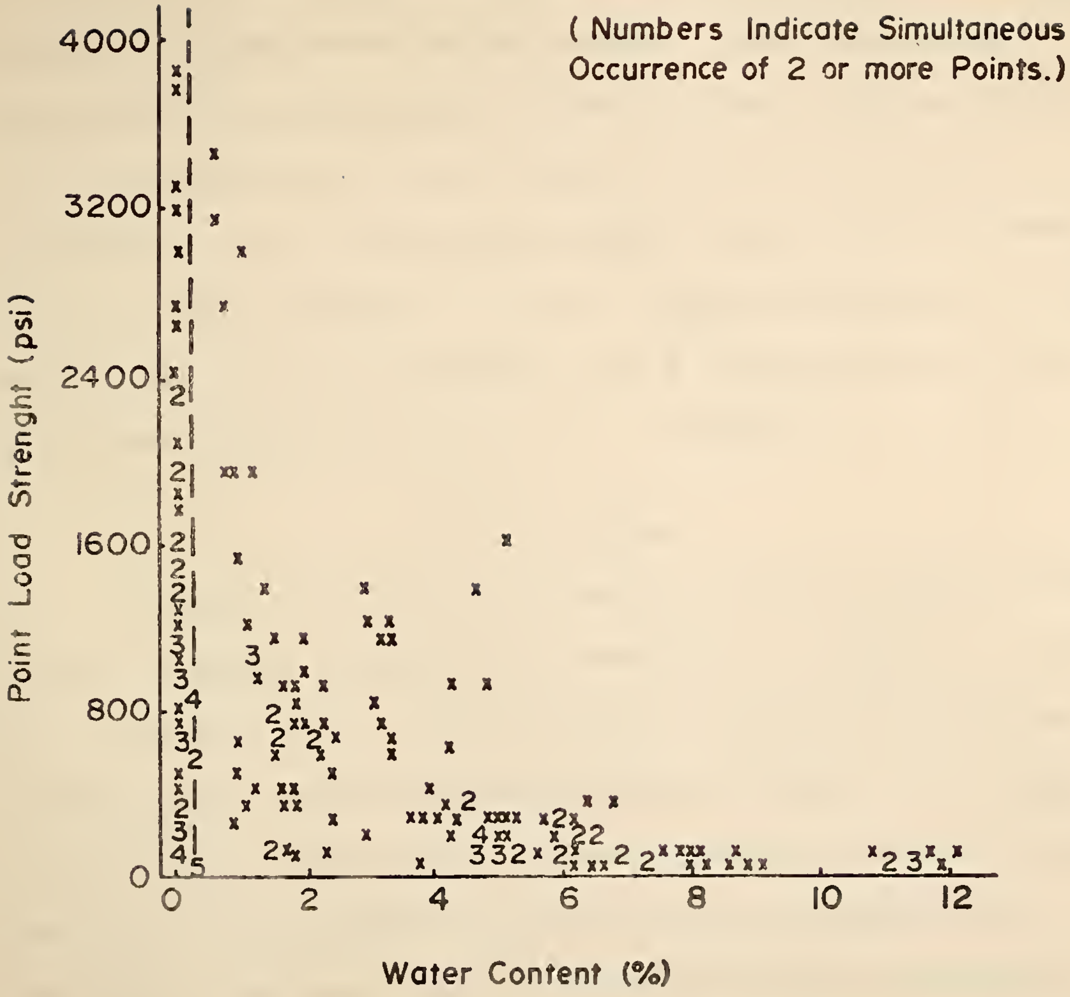


FIGURE 50 VARIATION IN POINT LOAD STRENGTH WITH WATER CONTENT FOR ALL SHALES TESTED.

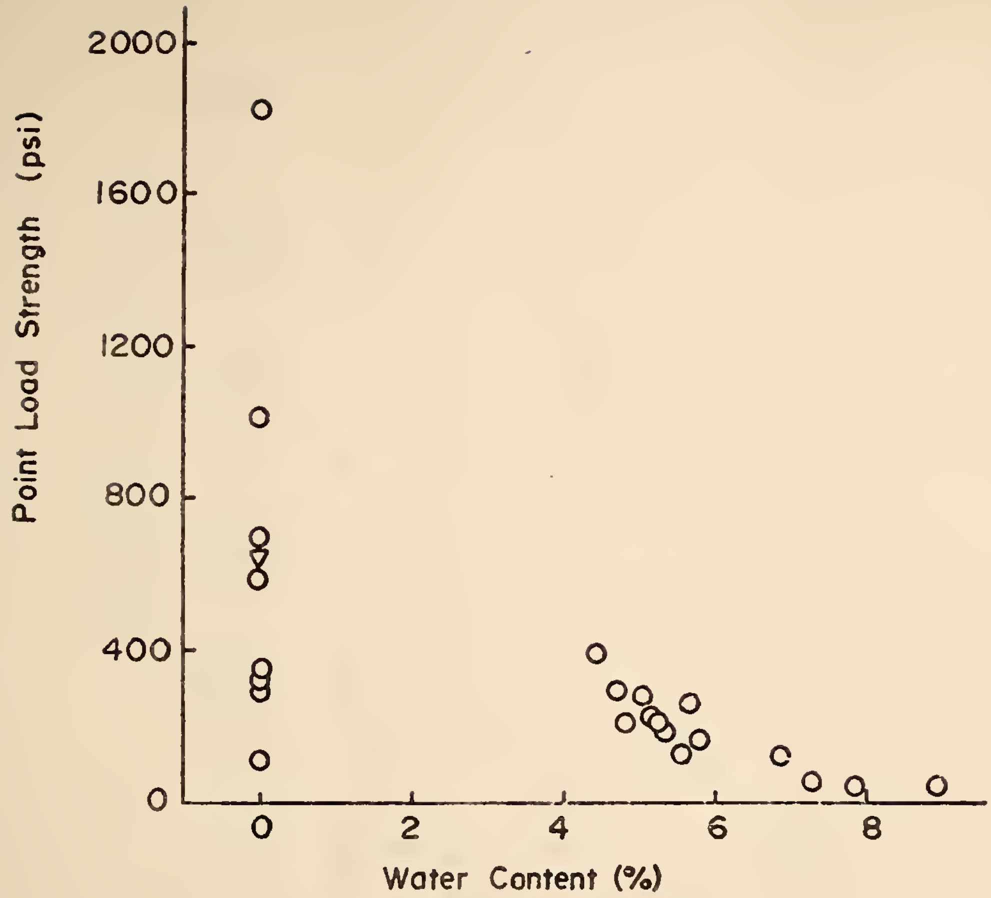
content test data, for Shales Nos. 2, 4, 11 and 20. Figures 51, 52, 53 and 54 show the test data and the results of simple regression analyses for Shales Nos. 2, 4, 11 and 20 respectively.

The data shown in Figures 51 through 54 illustrate the extensive variability in the oven-dried PLS samples. Linear regressions performed on these four shales resulted in higher R^2 values for the $w > 0$ condition than for $w \geq 0$. Using a mean value of the $w = 0$ results with the $w > 0$ test data resulted in still higher R^2 values for Shales Nos. 2, 11 and 20. The $w > 0$ and mean value for $w = 0$ data analysis for Shale No. 20 resulted in an R^2 value greater than the $w \geq 0$ condition, but less than the $w > 0$ condition.

5.2.4 Conclusions

As expected, the PLS test provided a relatively quick and convenient means of assessing the strength of shale pieces. Although the test is not extensively used, and lacks a uniformly agreed upon theoretical grounding, it appears sensitive to small variations in the strength of shale pieces. This enables strength testing of relatively weak and non-durable shales which are not amenable to more conventional laboratory procedures. The tests described required no particular sample preparation.

The PLS test results obtained in this investigation showed a wide scatter for shales tested after oven drying. Three possible explanations for this are indicated in the preceding section. About one-half of the data acquired from non-oven dried PLS specimens show fairly good linear correlations with shale moisture content as the independent variable. The combined results of all PLS tests show a definite trend of

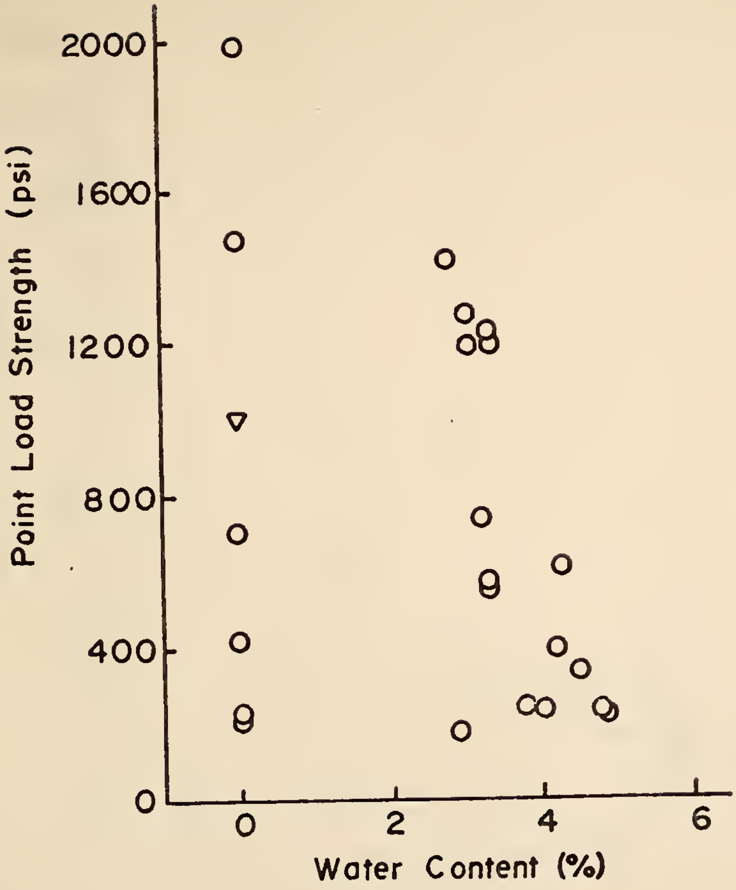


▽ Indicates Mean Value of $w = 0$ Tests

Results of Linear Regressions:

Analysis	Equation	R^2
$w > 0$	$PLS = 596 - 70.0w$	0.75
$w \geq 0$	$PLS = 642 - 76.3w$	0.35
▽ and $w > 0$	$PLS = 626 - 55.3w$	0.80

FIGURE 51 RESULTS OF PLS TESTS ON SHALE NO. 2.

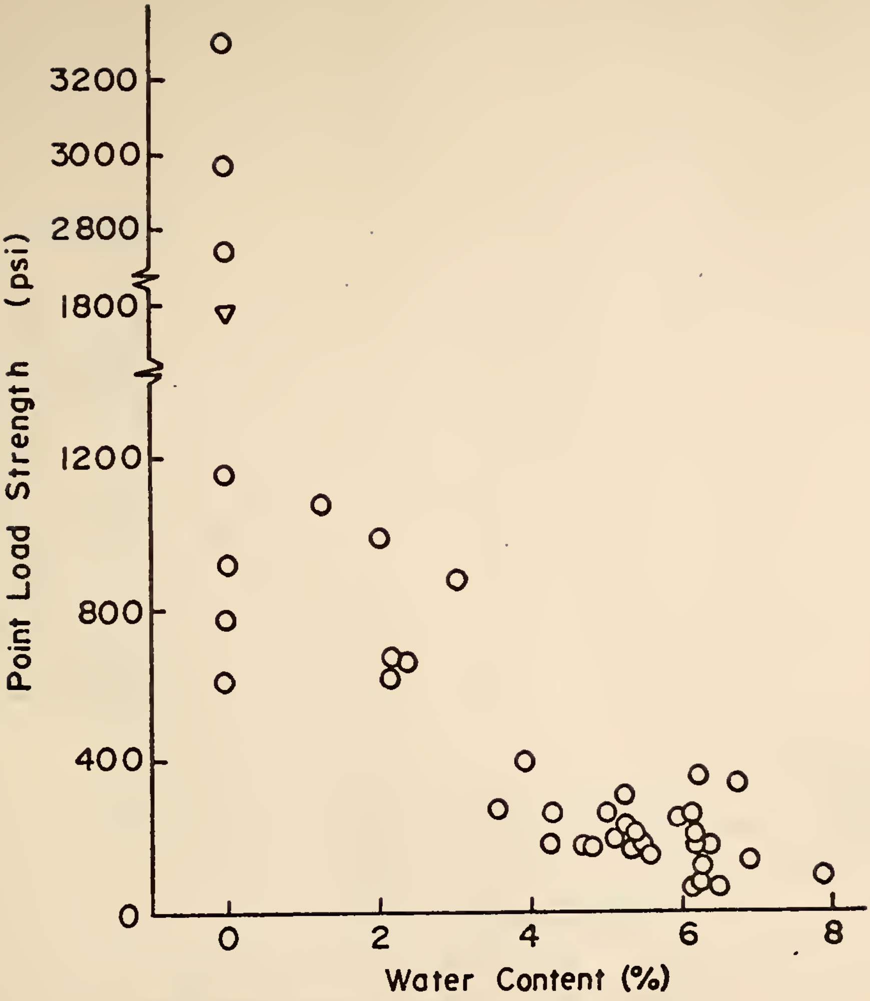


▽ Indicates Mean Value of w = 0 Tests

Results of Linear Regressions:

Analysis	Equation	R^2
w > 0	PLS = 2244-429 w	0.43
w ≥ 0	PLS = 951-87.6 w	0.09
▽ and w > 0	PLS = 1420-212 w	0.28

FIGURE 52 RESULTS OF PLS TESTS ON SHALE NO. 4.

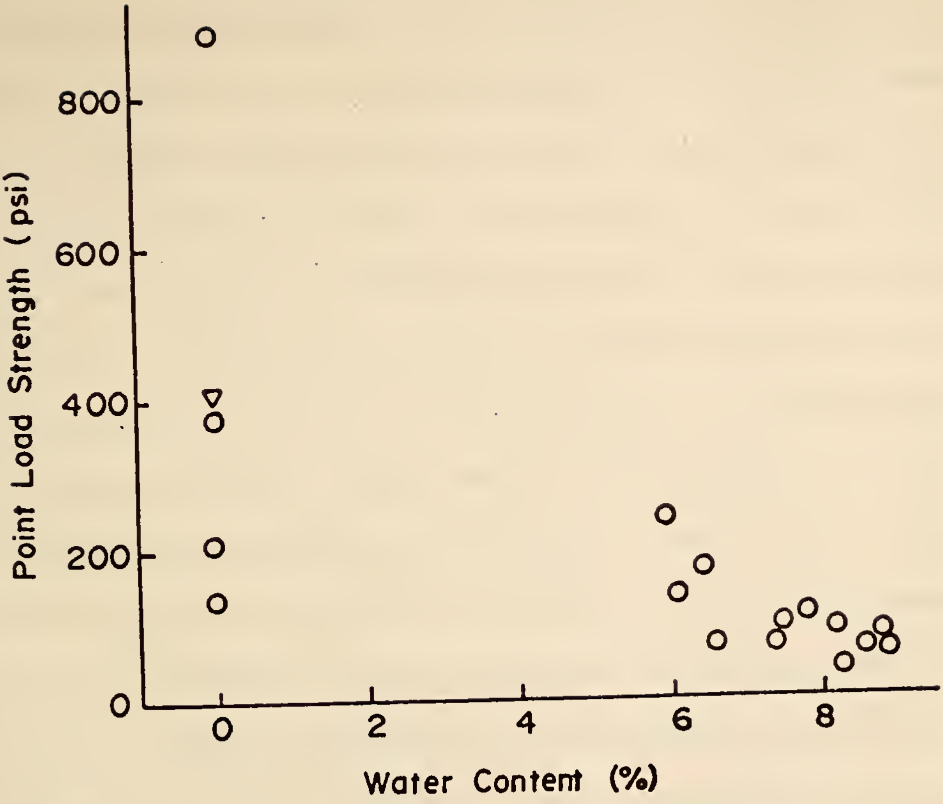


▽ Indicates Mean Value of w = 0 Tests

Results of Linear Regressions:

Analysis	Equation	R^2
$w > 0$	PLS = 997 - 136 w	0.71
$w \geq 0$	PLS = 1542 - 234 w	0.56
▽ and $w > 0$	PLS = 1189 - 171 w	0.74

FIGURE 53 RESULTS OF PLS TESTS ON SHALE NO. 11.



▽ Indicates Mean Value of $w = 0$ Tests

Results of Linear Regressions :

Analysis	Equation	R^2
$w > 0$	$PLS = 400 - 39.4\,w$	0.56
$w \geq 0$	$PLS = 402 - 39.6\,w$	0.45
▽ and $w > 0$	$PLS = 404 - 39.8\,w$	0.87

FIGURE 54 RESULTS OF PLS TESTS ON SHALE NO. 20.

decreasing strength with increasing moisture content. This trend agrees with reports from other investigations (7, 9, 63, 84), and with the results of Scleroscope hardness tests previously described.

The point load strength of shale pieces can be quickly and easily measured. PLS tests can be performed on relatively small pieces of shale, hand-collected specimens or pieces of drilled rock core, enabling the results of a number of PLS tests to be conveniently obtained and subjected to statistical analysis. The natural non-homogeneity of most shales requires such statistical analysis of data, whether they are to be used for classification or design purposes.

In summation, point load strength testing appears to be an adequate method of measuring the strength of shale pieces. This strength measure may prove to be correlatable with such embankment construction parameters as ease of excavation and degradation due to compaction. Further investigation of these aspects should prove beneficial to the state of the art of shale embankment construction.

6. SOAKING DEGRADATION AND ABSORPTION MEASUREMENTS

6.1 Introduction

Deo (23) and Chapman (16) have described the poor durability characteristics of many Indiana shales and the consequent effects on embankment stability. Various slaking tests and other experimental procedures have been reported by these two authors in an attempt to classify Indiana shales by durability, as a means of improving embankment design. However there are no recorded observations of the actual changes in particle gradation or strength due to long term slaking of shales in service. Neither are there any quantitative measures of the effects of slaking on the stability of compacted shale embankments. Ultimately such information will be needed from actual field tests before the state of the art of shale embankment design can be significantly advanced.

Ideally an experimental field test program could include the following observations, to be made over some extended period of time.

1. Extent of saturation within the embankment and amount of water absorbed by individual shale pieces.
2. Change in gradation of the compacted shale.
3. Change in shear strength of the compacted shale.
4. Magnitude of internal settlements and embankment slope movements.

Concurrent laboratory studies could include measurements of the effect

of moisture on degradation, and the combined effects of degradation and moisture on the strength and compressibility of compacted shale samples. Deo (23) has already reported results of CBR (compressibility) tests on as-compacted and soaked samples of fifteen Indiana shales.

Although somewhat beyond the scope of degradation due to compaction and strength and hardness testing, this investigation attempted to address two questions relevant to the problem of shale-moisture interaction. Tests were performed to measure the degradation due to slaking of two shales over an extended period of time. Additionally, observations were made on the distribution of adsorbed and absorbed moisture in soaked shale samples and in samples prepared for Proctor type compaction tests. These tests can be considered indicative of the results of adding moisture to improve field compaction of shales, and of saturation of shales already degraded by the compaction process.

6.2 Test Procedure

Thirteen one pound samples each of Shale Nos. 11 and 14 were prepared with the "A" gradation and placed in a loose condition in glass jars. The jars were filled with distilled water to a depth of about one inch above the surface of the shale pieces. Each jar was then closed with a screw type lid and sealed with rubber cement. The samples were then stored at room temperature where they could not be disturbed.

One jar of each shale type was opened and wet sieved at intervals of one to four weeks. The results of the wet sieving were used to calculate the particle degradation parameters previously described, for each sample.

The wet sieving operation was performed as follows.

1. Seven U.S. Standard sieves (Nos. 1/2", 3/8", 4, 8, 16, 30, and 50 were arranged in a single stack.

2. The sample jar was opened and the contents poured through the sieve stack.

3. The shale particles were washed through the sieve stack using about two gallons of water poured by hand into the top sieve. The wash water was also used to thoroughly rinse out the sample jar to insure removal of all visible sample fines.

4. After about one half of the wash water had been poured through the sieves, the sieve stack was gently shaken by hand. The remaining water was then poured through the stack. During both washing stages an attempt was made to pour uniformly over the entire area of the top sieve.

5. The sieves were separated and a moisture content sample was obtained from the material retained on each sieve. The sieves were placed in ceramic bowls and oven dried at 105° C until a constant weight was recorded.

6. The weight of oven dried material retained on each sieve and the weight of material removed to the moisture content tares was added, and the percentage of each fraction determined.

The sample fractions were then used to determine the IC, AA and PA terms for each sample. Because of clogging problems, the Nos. 100 and 200 sieves were not used, and their respective terms were not included in determination of the degradation parameters.

As mentioned previously, a sample of each shale was opened and

sieved at intervals of about one to four weeks. The last samples were sieved after soaking for 218 days (Shale No. 11) and 226 days (Shale No. 14), i.e. about seven and one half months. Test results are discussed in the following section. The degradation parameters and soaking time for the two shales are tabulated in Appendix F.

Inspection of the moisture contents measured for the various fractions of the long-term soaked samples indicated a fairly regular increase in the relative moisture content with decreasing particle size. From this observation, a question arose as to the distribution of water added to shale samples to improve their densification characteristics. The effects of sample moisture and curing time have already been discussed, however further tests were performed to measure moisture distribution within Proctor type compaction samples as described below.

Eight one pound samples of Shale No. 11 were spread out in flat pans and allowed to air dry at room temperature for 24 hours. Water was sprayed on the samples and each was mixed by hand with a small spatula. The samples were not cured, but mixed until it appeared that uniform moisture distribution had been achieved. The uniformity of moisture distribution was only qualitatively evaluated by observation of the color of the shale. The wetted shale was noticeably darker than the dry material. Two moisture content samples of about 50 grams each were obtained from the sample, and the remainder of the material was separated over a No. 4 sieve. Sieving was performed using a mechanical sieve shaking apparatus which was operated for at least one minute per sample. Following sieving, the shale was placed in tares

and separate moisture content determinations made for the material which passed or was retained on the sieve.

The No. 4 sieve was used in these tests because the moisture tended to bind the finer shale particles into lumps. These "mud" lumps would not pass sieves finer than the No. 4 size. The results of this test are tabulated and discussed in the following section.

6.3 Test Results

The long-term soaking degradation tests were completed for twelve samples of Shale No. 11 and thirteen samples of Shale No. 14. Regression analyses were performed to determine the relation between the three degradation parameters and the soaking time for each shale, and for both shales together.

Using the "t" statistic and a 99% reliability level, minimum significant values of Pearson's correlation coefficient were calculated and compared with the soaking time vs. degradation correlations. There were no significant correlations for the results of tests on either shale. Figure 55 shows the degradation due to slaking as measured by the IC parameter for both shales. Similar plots could be drawn using the AA or PA terms, however the same variability is present in those data also.

The sample degradations were not influenced by any sort of physical disturbance prior to unsealing each specimen jar. The samples were stored in such a way as to insure that degradation resulted only from the slaking effect of water. Samples were not handled, exposed to significant changes in temperature, or removed from water until the time of testing. Although not under load, it was felt that the sample

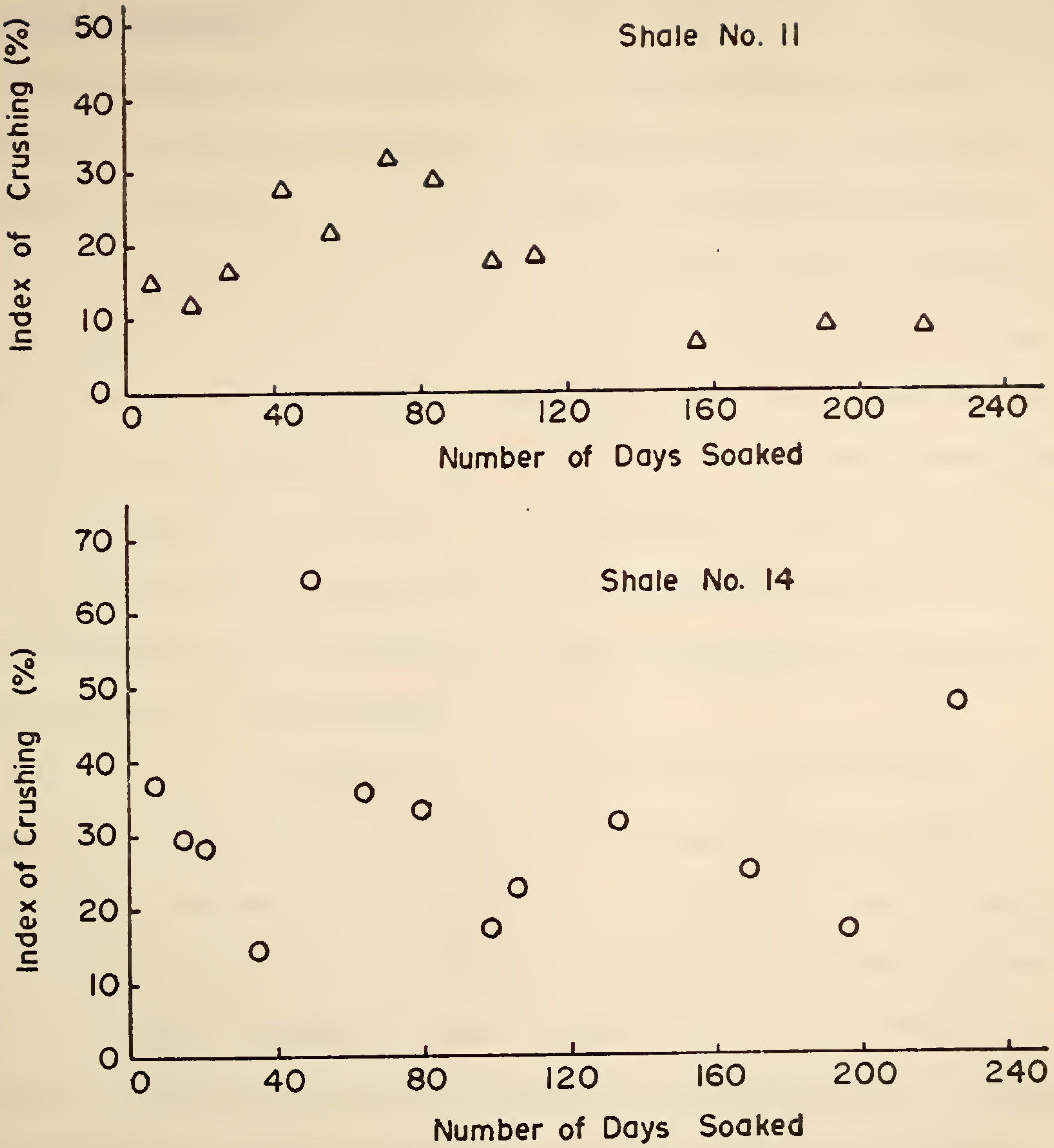


FIGURE 55 DEGRADATION (IC) OF TWO SHALES SUBJECTED TO LONG TERM SOAKING.

deterioration would closely parallel that of shale particles in a saturated embankment.

The irregular slaking degradation that was observed over the period of about seven and one half months may be due to variations in the material tested, i.e. the shale samples' natural nonhomogeneity. The period of observation may have been too short to allow development of a degradation trend greater than the individual variability of the samples tested. Alternatively it can be concluded that slaking is not a time dependent phenomenon, or that the test procedure was insensitive to the degradation that occured. As previous work reported in the literature does not include tests of the type described here, it is assumed that more data are needed to enable developement of a rate-of-slaking model for Indiana shales.

Although the degradation of the soaked samples varied widely, the amount of water absorbed by the shale samples appeared to be fairly consistent. Samples of both Shale Nos. 11 and 14 were found to have significantly greater water contents for particles of decreasing size. This was expected because the larger surface area per unit weight of the finer particles enabled the absorption and adsorption of a relatively larger volume of water per particle, as the particle size decreased. The measured water contents for various particle size ranges are shown in Table 13 for samples of the two shales after soaking for one week. The two shales had remarkably similar values of water content for the various particle size ranges.

The water contents of the two shale types showed some variation over the seven and one half month period of observation, but this was

Table 13.

Water Content of Various Fractions of Two Shales After Seven Days Soaking.

Sieves Defining Particle Size Range	Water Content of Shale No. 11 (percent)	Water Content of Shale No. 14 (percent)
3/4" to 1/2"	10.7	10.2
1/2" to 3/8"	10.9	10.7
3/8" to #4	17.7	17.6
#4 to #8	22.8	26.9
#8 to #16	47.7	47.7
#16 to #30	59.1	59.0
#30 to #50	61.8	62.3

fairly minor and did not appear significant compared to the variation in the degradation terms. Figure 56 shows the change in water content of three of the particle size ranges (3/4" to 1/2", No. 4 sieve to No. 8 sieve, and No. 30 sieve to No. 50 sieve) throughout the test period. The other particle size groups showed similar moisture content variations, but were omitted from Figure 56 for the sake of simplicity.

As shown in Figure 56 there was some variation in the amount of water absorbed and adsorbed by the different shale samples, but the water content distribution with respect to particle size was fairly constant. The data show a slight trend in increasing water content, for the various fractions, as soaking time was increased. This trend is small with respect to the individual sample variations however.

Considering the largest particles which were included in these test specimens (i.e. passing the 3/4 inch sieve and retained on the 1/2 inch sieve), the data show that samples soaked at least 100 days had an average water content of about 10% for Shale No. 11, and 14% for Shale No. 14. The samples had initial average water contents of 4.5% and 5.1% respectively, before soaking was started.

As described in the preceding section, eight samples of Shale No. 11 were mixed with various amounts of water and separated over the No. 4 sieve. The results of water content measurements for seven of these samples are shown in Table 14. As these tests were performed, it was found that increasing moisture tended to bind the finer particles together into lumps, and to cause the fines to adhere to the larger particles. After the water content had reached about 13% the fines could no longer be separated from the particles retained on the No. 4 sieve

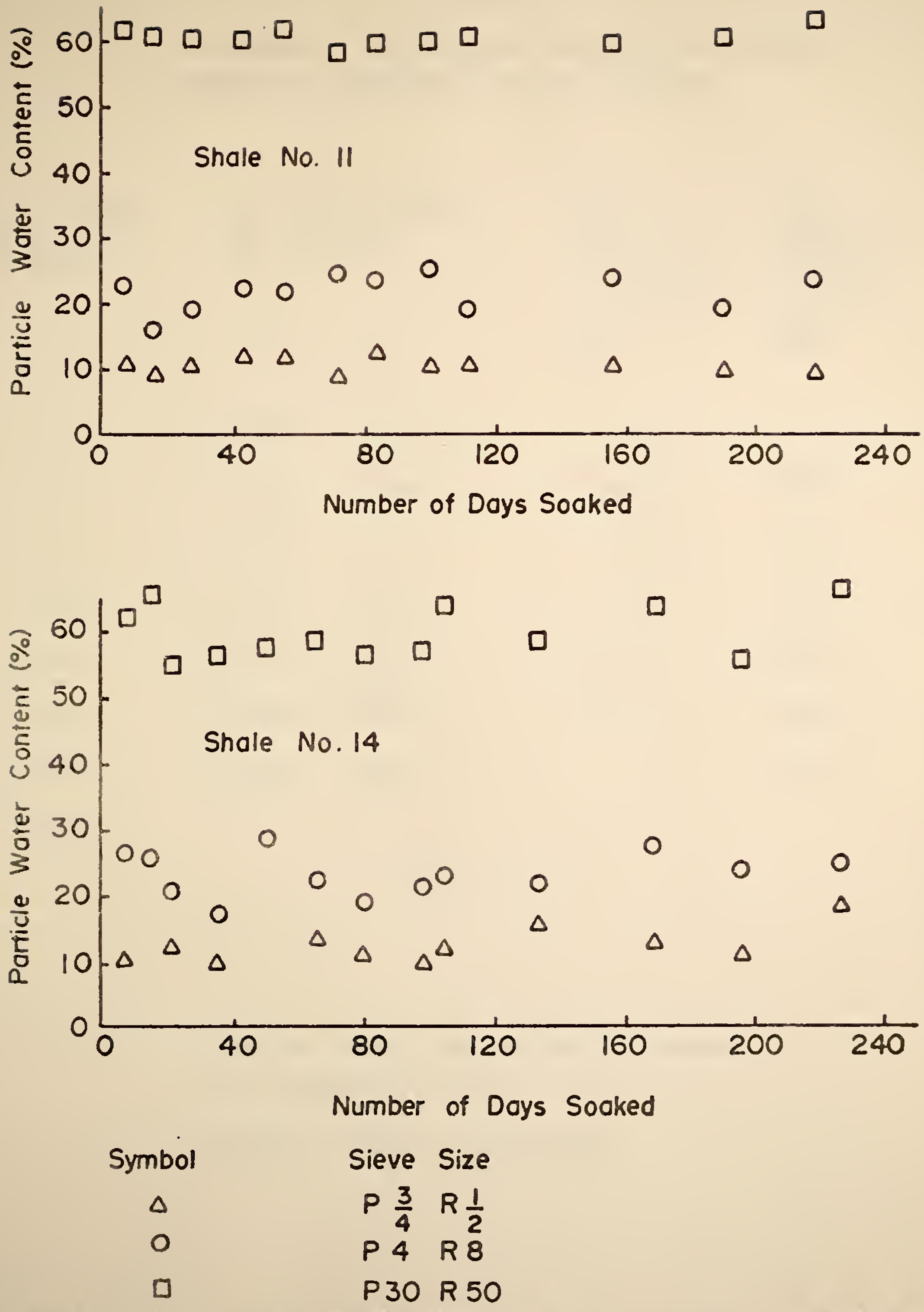


FIGURE 56 VARIATION IN PARTICLE WATER CONTENT WITH SOAKING TIME FOR TWO SHALES.

Table 14.

Water Content of Specimens Prepared For Proctor Compaction, Divided By The No. 4 Sieve.

Water Added To One Pound Samples	Bulk Water Content (Two Samples)		Water Content of Material Retained On No. 4 Sieve	Water Content of Material Passing No. 4 Sieve
none	0.9%	1.0%	0.9%	1.0%
11 ml	3.3%	3.2%	3.1%	3.3%
23 ml	6.4%	-	4.8%	6.9%
34 ml	7.8%	8.1%	7.5%	12.0%
45 ml	10.7%	11.2%	11.5%	13.1%
57 ml	12.7%	12.1%	12.0%	14.9%
68 ml	14.3%	13.3%	*	15.4%
79 ml	*	*	*	*

* Indicates Sample Was Too Wet To Separate
See Discussion In Text

- Indicates Sample Was Spilled

by mechanical shaking. As a result, no moisture content determinations were made for the wettest sample, and no determination was made for material retained on the No. 4 sieve for the second wettest sample. These missing data are indicated by asterisks in Table 14.

The data shown in Table 14 indicate that as the moisture content of the total sample increases, the finer particles tend to absorb a relatively larger amount of water than do the coarse particles. This is shown graphically in Figure 57.

The data shown in Table 14 for the two particle size groups, as well as the variation in the bulk moisture content indicate that the water content of any shale sample is at least partially dependent on the sample's gradation. This is true for both the Proctor type samples and the soaked samples. Figure 7 (Section 2.2) shows a fairly constant trend between the AGM and water content, for samples obtained from a shale test pad.

6.4 Conclusions

Although the soaking tests discussed above represent an imperfect model of field behavior, several conclusions can be drawn from the observed data.

The particle degradation due to slaking of samples soaked up to seven and one half months does not appear to be a time dependent phenomenon. Within the constraints of the test, no degradation trend could be determined which exceeded the variability in results of the individual samples.

The soaked shale samples showed a significant range in water

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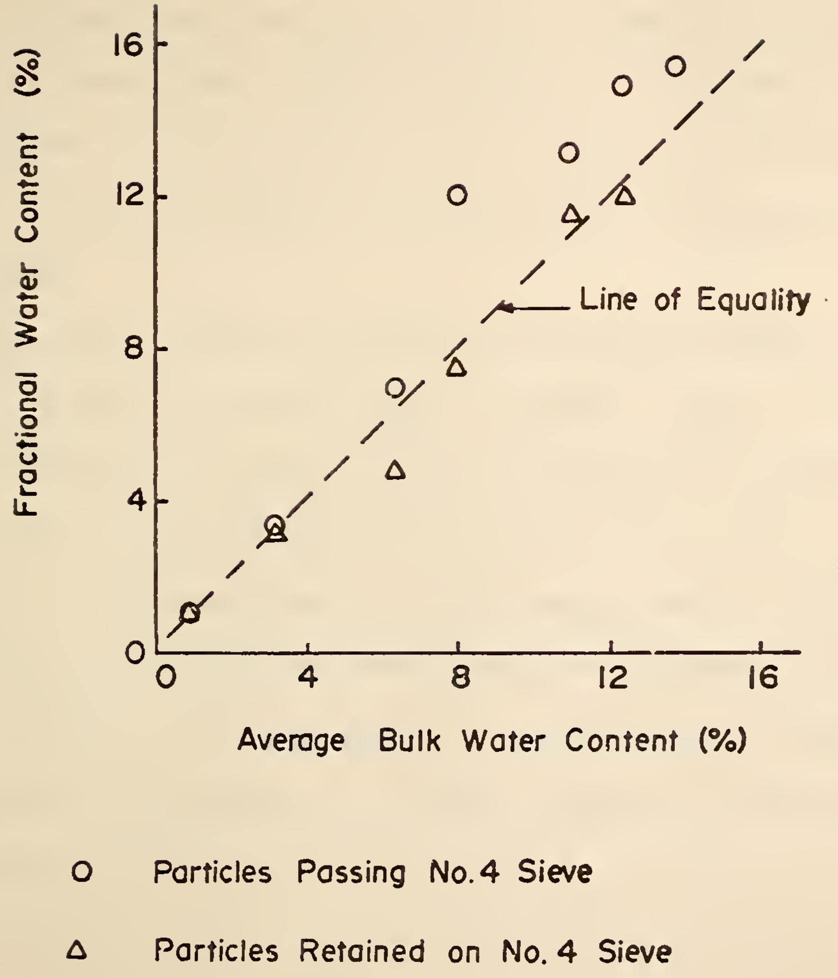

FIGURE 57 FRACTIONAL WATER CONTENT OF SPECIMENS PREPARED FOR PROCTOR COMPACTION, DIVIDED ON NO. 4 SIEVE.

content for particles of various sizes. Fractional water contents were found to increase as the particle size decreased. The water content of particles of any particular size tended to increase only slightly as the soaking time increased.

Fractional water contents of samples prepared for Proctor type compaction tests showed a similar variation. As the total water content was increased, particles passing the No. 4 sieve showed a greater relative increase in moisture content than did coarser particles in the same sample.

From these results it appears that the water content of any shale sample is influenced by that sample's gradation. Some care is thus needed in relating the water content of laboratory specimens to the water content of coarser materials compacted in the field. Furthermore it appears that adding water to a shale sample, even by soaking, does not appreciably increase the water content of pieces one half inch in size or larger. This may be a point of some concern in future testing of shales where the influence of sample moisture is to be controlled or measured.

7. SUMMARY AND RECOMMENDATIONS FOR FURTHER RESEARCH

The conclusions and recommendations presented in this chapter are based on the results of the previously described laboratory tests, a review of current engineering literature, and a limited amount of information from field test pads.

The most outstanding characteristic of the current state of the art in shale embankment construction is the minimal quantity of reliable field information. Although several agencies have performed field tests during construction of shale embankments for highways and dams, the resulting information has not been properly correlated or disseminated throughout the civil engineering profession. There are several published statements about the need for improving construction practices however.

It is hoped that the results of this investigation will assist in improving the preconstruction evaluation techniques available to engineers working with shale fills. It is also hoped that this investigation will encourage engineers to increasingly use field test pads, long term observations of shale embankment performance, statistical validation of data collected and improved reporting of both successes and failures.

The lack of "real world" information is keenly evident in evaluating the results of the laboratory tests described in this report.

Detailed conclusions are cited at several points in this report.

Specific conclusions may be found in the sections listed below.

Kneading compaction tests, Section 4.1.4, page 77.

Gyratory compaction tests, Section 4.2.5, page 104.

Static compaction tests, Section 4.3.4, page 117.

Impact compaction tests, Section 4.4.4, page 134.

Scleroscope hardness tests, Section 5.1.4, page 154.

Point load strength tests, Section 5.2.4, page 168.

Soaking degradation and absorbtion tests, Section 6.4, page 185.

In general it appears that shale compaction and degradation are interrelated both during construction and during the service life of an embankment. Both densification and particle degradation appear to be self-limiting reactions, regardless of compaction effort, moisture content, or initial gradation. However these same three factors cannot be carried to extremes in field work (as in the laboratory), and further attention to their effects is needed.

In comparing the results of kneading, gyratory, static, and impact compaction methods, the static compaction tests produced the best relation between compaction effort, particle degradation and compacted unit weight. The static test appears suitable as a compaction-degradation index, but more work is required to explore the applicability of its results.

Of the three degradation analysis parameters used in this investigation, the index of crushing (IC) repeatedly proved to be more sensitive, and to afford better correlations with compaction effort than either the arithmetic or percent change in aggregate gradation modulus.

Both the Scleroscope hardness and point load strength (PLS) tests showed the significant effects of moisture content on the integrity of shales. Because of its ease of performance and minimal sample preparation, the PLS test is recommended for future use in characterising both the strength and homogeneity of shale rocks. Future research may find significant correlations between the PLS and other shale properties, that could lead to its adoption as a classification or design test.

Finally, the soaking degradation, Proctor compaction and shale absorption tests must be considered as proof that the effect of moisture on shale, and the use of laboratory tests to simulate field behavior, are more complex than previously considered. Future research might profitably be directed towards investigation of the physical-chemical aspects of the slaking phenomenon, as well as towards a better understanding of the moisture-density relation for compaction of shales.

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APPENDICES

APPENDIX A

EXPERIMENTAL MATERIALS

Results of tests on twenty-two samples from twelve Indiana shale formations are described in this report. The samples range in origin from Ordovician through the Pennsylvanian systems, covering the entire geologic column presently exposed in Indiana. Harrison and Murray (31) and Deo (23) have discussed pertinent aspects of the geology of these shales.

Except for Shale No. 13, all of the shale samples available for this investigation were provided by, or obtained in cooperation with, employees of the Indiana State Highway Commission. In addition, the results of a large number of shale property tests were provided by the ISHC (39). This cooperation is gratefully acknowledged.

Chapman (16) has described the storage conditions adopted for the six shales used in his investigation. The use of steel cans with plastic liners has proven to be convenient, but unsatisfactory for maintaining a constant moisture content in the shale. The variation in moisture content from the reported initial value, for periods of up to almost 35 months, is shown in Figure 58. Proceeding clockwise in Figure 58 from the upper left hand corner the figures shown are for the shales numbered: 3, 4, 6, 5, 2 and 1, by Chapman. Considering the influence of moisture content on the properties of shales discussed elsewhere in this report, it is obvious that improved methods

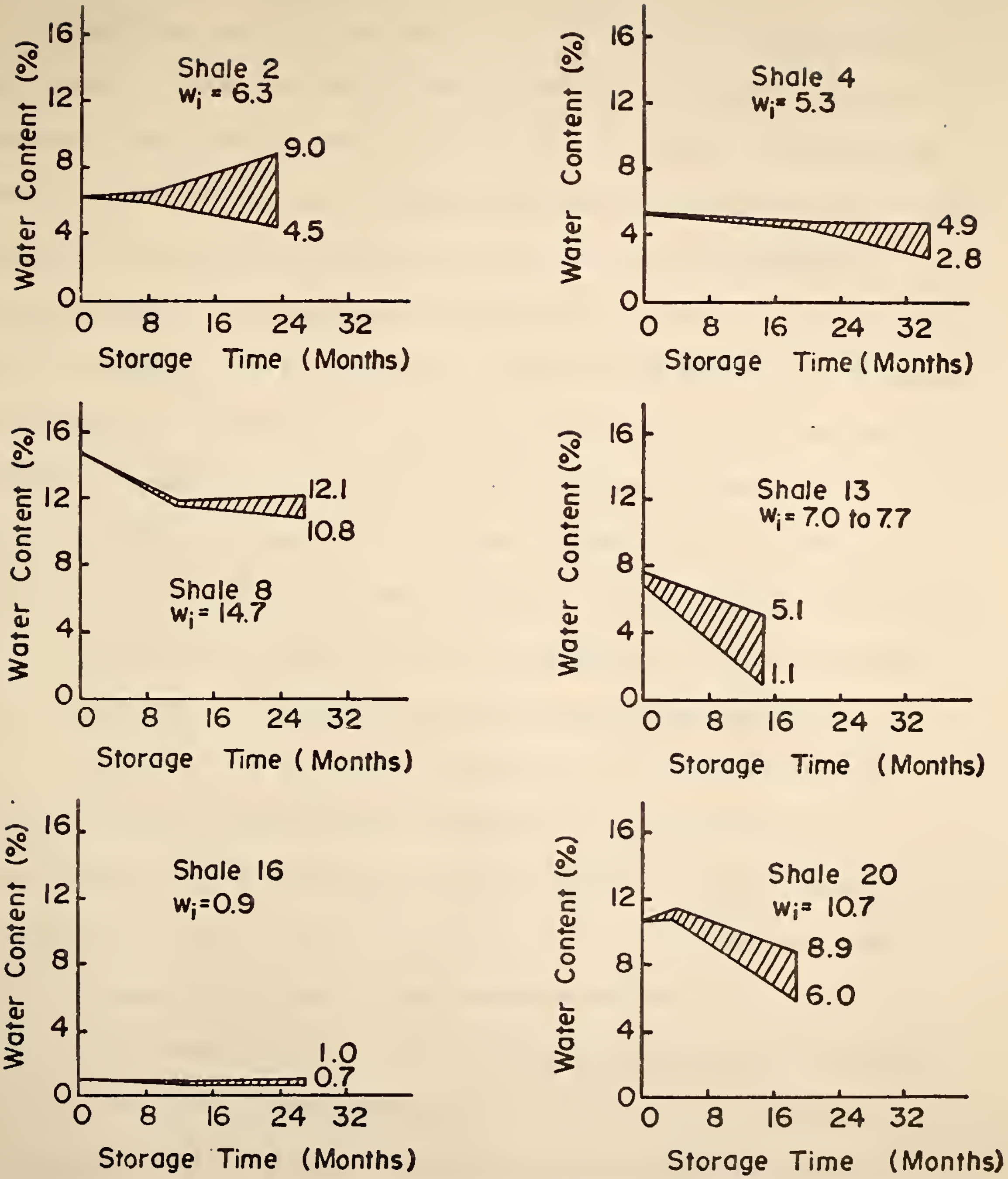

FIGURE 58 VARIATION IN MOISTURE CONTENT OF SHALE SAMPLES STORED IN STEEL CANS WITH PLASTIC LINERS.

of sample storage are needed.

This investigation used Shale No. 11, the "Attica model shale", as a means of comparing the effects of several different compaction methods. About 4000 pounds of Shale No. 11 was stored in a steel bin with a plywood cover. Inside the bin the shale was covered with plastic sheets. The plywood cover, which was painted with a waterproof varnish to resist warping, was sealed closed with plastic tape except when the material was actually being removed from the bin for testing. Despite this care, the water content range of Shale No. 11 varied from 5.2 to 6.6%, to, 1.2 to 7.9% over a period of twelve months. A similar quantity of Shale No. 14 showed a water content variation of 3.0 to 3.6%, to, 0.8 to 2.5% over a period of eight months, when stored in the same way. Both bins were kept indoors, under fairly constant room temperature and humidity conditions, during this period.

Shale Nos. 1, 3, 5, 6, 7, 9, 10, 12, 17, 18, 19, and 21 were received from the ISHC Division of Materials and Tests sealed in plastic bags. These samples were prepared for Scleroscope Hardness testing and were oven dried during the tests. No attempt was made to control or monitor contents before hardness testing.

As mentioned previously, all but one of the shales tested were obtained from, or in cooperation with, the ISHC. Most of these shales were sampled during ongoing construction work, however a few were obtained from exploratory drilling operations and from privately owned quarries. In the following descriptions, the seven digit number in parentheses refers to the sample number assigned to the material by the ISHC Division of Materials and Tests. Information on the origin,

use, classification, and properties of the material can be correlated through this number (39).

Shale Nos. 1 and 2 (74-54878, 74-54684) are from the Mansfield formation occuring at the Pennsylvania-Mississippian unconformity. Both shales were sampled in Perry County as part of embankment constuction operations. These shales are gray colored, flaggy, and are classified as soil-like. Shale No. 2 is the third shale described by Chapman (16).

Shale No. 3 (75-55653) is from the Clore formation of the Mississippian Chester series of sediments. This shale was also used in embankment construction in Perry County. Shale No. 3 is dark gray, flaky, and is classified as soil-like.

Shale Nos. 4, 5 and 6 (74-54716, 74-54836, 75-55044) are from the Palestine formation of the Mississippian Chester series. All three were sampled in Perry County and are soil-like. Shale No. 4 is the fourth shale described by Chapman (16). Shale Nos. 4 and 5 are dark gray or black, and are flaky textured. Shale No. 6 is light to medium gray, and flaggy.

Shale No. 7 (74-54767) is a relatively soft, gray, soil-like material. This shale is from the Waltersburg formation of the Chester series. This shale was used as subgrade material on a highway project in Crawford County.

Shale No. 8 (73-51703) was one of the first shales to be investigated at Purdue. Samples were obtained from the lower portion of the Hardinsburg formation in Perry County. It is a flaggy, gray shale, also from the Mississippian Chester series. This shale is No. 1

in the Chapman report (16).

Shale No. 9 (74-54973) is from the Big Clifty formation of the Chester series. Samples of the flaggy, gray, soil-like shale were obtained from rock core borings in Orange County.

Shale No. 10 (74-54972) is from the predominantly limestone Haney formation of the Chester series. This shale is unusual in that is yellow-tan in color, and has a very low natural moisture content (0.9%). This shale is flaggy and soil-like. Samples were taken from core borings in Orange County.

Shale Nos. 11 and 12 (75-55564, 75-55316) are from the Borden group of sediments from the Mississippian age Valmeyeran series. Both shales were obtained from a disused quarry near the town of Attica in Fountain County. Shale No. 11 is the "Attica model shale" used as a control material for comparing the results of various compaction tests in this study. This material is dark gray, flaky, and quite fossiliferous. Unweathered samples were excavated with a rubber-tired front-end loader. Shale No. 12 is flaggy and somewhat less durable than Shale No. 11. Shale No. 12 was sampled by hand. Both shales are soil-like.

Shale No. 13 (75-55315) was obtained from an abandoned quarry in Tippecanoe County. This shale is also from the Borden group of the Mississippian Valmeyeran series, but is much harder and more durable than the Attica shale. Shale No. 13 is lightbrown or tan in color, fairly massive (depending on the degree of weathering) and is classified as rock-like. This material was very difficult to excavate by hand with pick and shovel. Shale No. 13 is the sixth shale

described by Chapman (16).

Shale Nos. 14 and 15 (75-55721, 75-55505) are from the New Providence formation of the Valmeyeran series. Both shales were obtained from a highway excavation in Floyd County. Excavation was accomplished by bulldozer assisted motor scrapers. Shale No. 14 is light gray colored, massive, and is classified as soil-like. About 4000 pounds of Shale No. 14 was obtained for use as the control material ("New Providence model shale") for other research currently underway. Shale No. 15 is a gray, flaky, soil-like material.

Shale Nos. 16, 17, 18, and 19 (74-54621, 75-55718, 75-55486, 75-55487) are from the New Albany formation which occurs at the Mississippian-Devonian contact. Shale No. 15 is from the upper New Albany formation of the Mississippian age Kinderhook series. This shale is hard and massive, very dark gray or black, and has a very low (0.9%) natural moisture content. Shale No. 16 was obtained from an embankment construction project in Floyd County. Samples of this shale may contain crystals of pyrite or marcasite up to about one half inch in size. Shale No. 16 is classified as rock-like, and is the second shale described by Chapman (16).

Shale No. 17 is from the Selmier member of the lower New Albany formation of the Seneca series of Devonian age sediments. This soil-like material was obtained from a quarry in Marion County. Shale No. 17 is green or gray colored and flaky. Shale Nos. 18 and 19 are from the Blocher and Morgan Trail members of the lower New Albany formation, respectively. These two shales are gray colored, massive and are classified as rock-like. Shale Nos. 18 and 19 are also from Marion

County.

Shale No. 20 (75-55018) is a gray-green, flaggy material from the Kope formation of the Ordovician system, Cincinnatian series. This soil-like shale was the fifth shale described by Chapman (16).

Shale No. 21 (75-55291) is from the Dillsboro formation of the Cincinnatian series. It is a gray, soil-like shale. Both Shale Nos. 20 and 21 were sampled in Dearborn County.

Shale No. 22 (76-55014) was obtained during construction of an embankment compaction test pad in Union County, near the town of Liberty. This shale is gray, flaggy, and is classified as soil-like. Shale No. 22 is from the Whitewater formation of the Cincinnatian series of Ordovician rocks.

Further information on the properties of these twenty-two shales is shown in Table 15 (39, 40).

Table 15.

Properties of Experimental Materials. After ISHC (39, 40).

Shale Number	Natural Moisture Content (percent)	Natural Dry Density (pcf)	Five Cycle Slaking Index	Slake Durability Index 500 Rev. Dry	Slake Durability Index 500 Rev. Soaked	Modified Sodium Sulfate Soundness Test (% loss)	Standard Proctor Compaction $\gamma_{D\ max}$ (pcf)	Standard Proctor Compaction Optimum Moisture Content (%)
1	7.8	140.7	43.4	52.8	57.5	-	120.7	12.7
2	6.3	141.6	14.3	71.1	44.1	0	119.8	12.4
3	11.9	128.2	78.2	14.9	16.3	-	114.1	14.3
4	5.3	145.9	61.8	36.5	27.9	0	115.1	12.3
5	6.9	139.1	89.4	24.9	8.0	-	119.3	12.6
6	6.3	146.8	10.1	70.9	68.1	-	123.4	11.7
7	-	-	92.7	20.4	5.3	-	-	-
8	12.7	123.5	97.8	23.3	21.0	0	105.9	16.8
9	3.4	145.6	97.9	35.3	22.2	-	-	-
10	0.9	142.9	60.0	0.2	17.1	-	-	-
11	6.3	136.4	42.7	77.0	62.6	-	117.9	14.1
12	5.6	146.0	10.6	90.4	67.6	94.7	123.2	11.3
13	6.8	128.5	2.1	98.3	98.4	99.6	109.6	17.3
14	6.1	145.0	47.7	76.0	41.0	-	120.1	12.1
15	8.3	139.4	46.3	82.9	45.1	-	120.6	10.3
16	0.9	139.8	0	99.0	99.0	3.9	90.1	8.3
17	7.1	141.1	57.2	74.2	69.8	-	117.3	12.4
18	2.6	154.6	1.1	99.1	99.0	0.6	110.0	9.7
19	3.2	146.1	0.9	99.1	99.1	0.5	99.1	11.0
20	10.7	131.4	96.7	14.8	4.8	0	116.6	14.7
21	6.1	-	-	50.5	21.3	-	121.4	13.2
22	14.4	123.3	99.6	34.0	3.9	-	112.8	16.3

APPENDIX B

GENERAL LABORATORY PROCEDURES

The laboratory procedures used in this investigation have been mainly described in their relevant sections in the preceeding report. Several general laboratory procedures have not been previously described however, and are described here to assist future investigators in repeating or extending this work.

The shale samples used in the compaction tests were prepared by crushing fist sized pieces of rock. An electrically powered reciprocating jaw crusher, manufactured by the Denver Equipment Company was used to crush material to less than one inch in size. The crushed shale was scalped to minus 3/4 inch, which resulted in the "A" gradation bandwidth shown in Figure 12 in section 4.1.

All of the degradation test specimens were dry sieved before and after compaction. Typically the sieve stack consisted of ten U.S. Standard 8 inch sieves, arranged in decreasing order. The sieves used were 3/4 inch, 1/2 inch, 3/8 inch, and Nos. 4, 8, 16, 30, 50, 100, and 200. The sieves were arranged in two stacks of five sieves each, and the shale sample was sieved first from the 3/4 inch to a pan placed below the No. 8. The material passing the No. 8 sieve was then sieved through the No. 16 to No. 200 sieves with the minus No. 200 material collected in a second pan. Shaking of both stacks was accomplished with an electrically powered apparatus, operating at about five cycles per

second, for 30 seconds.

Each shale was sieved following the same procedure in order to minimize any variability in particle breakage caused by the sieve shaking. Because of the high slaking tendencies exhibited by the shales used in this investigation, all samples were dry sieved except as noted below.

Dry sieving sometimes may lead to erroneous gradation characterizations if the material is slightly damp. The presence of moisture may cause the formation of aggregates of fine particles which will be retained on some sieves with openings larger than the individual particles. Furthermore, there is the problem of very fine particles becoming suspended in air during sieve shaking. These fine particles may adhere to the sides or mesh of the sieves rather than passing through the mesh. No way to avoid these problems was discovered, however the error in measured gradation due to these effects was considered small compared to the effect of slaking which would occur during wet sieving.

The sole exception to the use of dry sieving in this investigation was in determining the final gradation of the long-term soaked degradation test specimens. These tests required shale samples to be submerged in water for several weeks or months. In these circumstances, it was felt that wet sieving would (a) cause no further slaking of the shales, and (b) be the only way to separate the cohesive "mud" (which resulted from the initial slaking) into individual particles.

Sample water contents were obtained by drying specimens in a 105° C oven to a constant weight. All water conten samples were weighed to the nearest 0.01 gram, before and after drying. Generally compaction

samples and other samples with a maximum particle size of about 1/2 inch reached a constant weight after three or four days. Larger pieces of shale took correspondingly longer with a maximum of 30 to 40 days required to dry some of the Scleroscope samples.

Significant problems were encountered in removing compaction samples from the various molds that were used. All of the compaction molds had removable base plates or were open ended sections of steel pipe. When samples had been only slightly compacted, the shale material could be extruded by gentle hand pushing through the mold and into a plastic sample bag. Densely compacted samples were also extruded by hand pushing, but several blows of a wooden mallet, applied around the outside of the mold were sometimes required to loosen the sample in the mold.

Very densely compacted samples could not be removed from their molds without apparent further degradation of the shale. Initial attempts to remove samples with a power driven extruding apparatus appeared to cause erratic degradation results for the compaction tests in question. This operation was thus discontinued and the compaction effort accordingly limited. No degradation test specimens were removed from their molds by chipping or digging.

NOTICE

The following Appendices listed in the Table of Contents of this Report have not been included in this copy of the Report:

Appendix	Title	Pages
C	Data from Shale Test Pads	210-211
D	Data from Compaction-Degradation Tests	212-222
E	Data from Strength and Hardness Tests	223-229
F	Data from Soaking Tests	230

A recipient of the Report may secure a copy of the Appendix desired upon request to:

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West Lafayette, Indiana 47907

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LEGEND

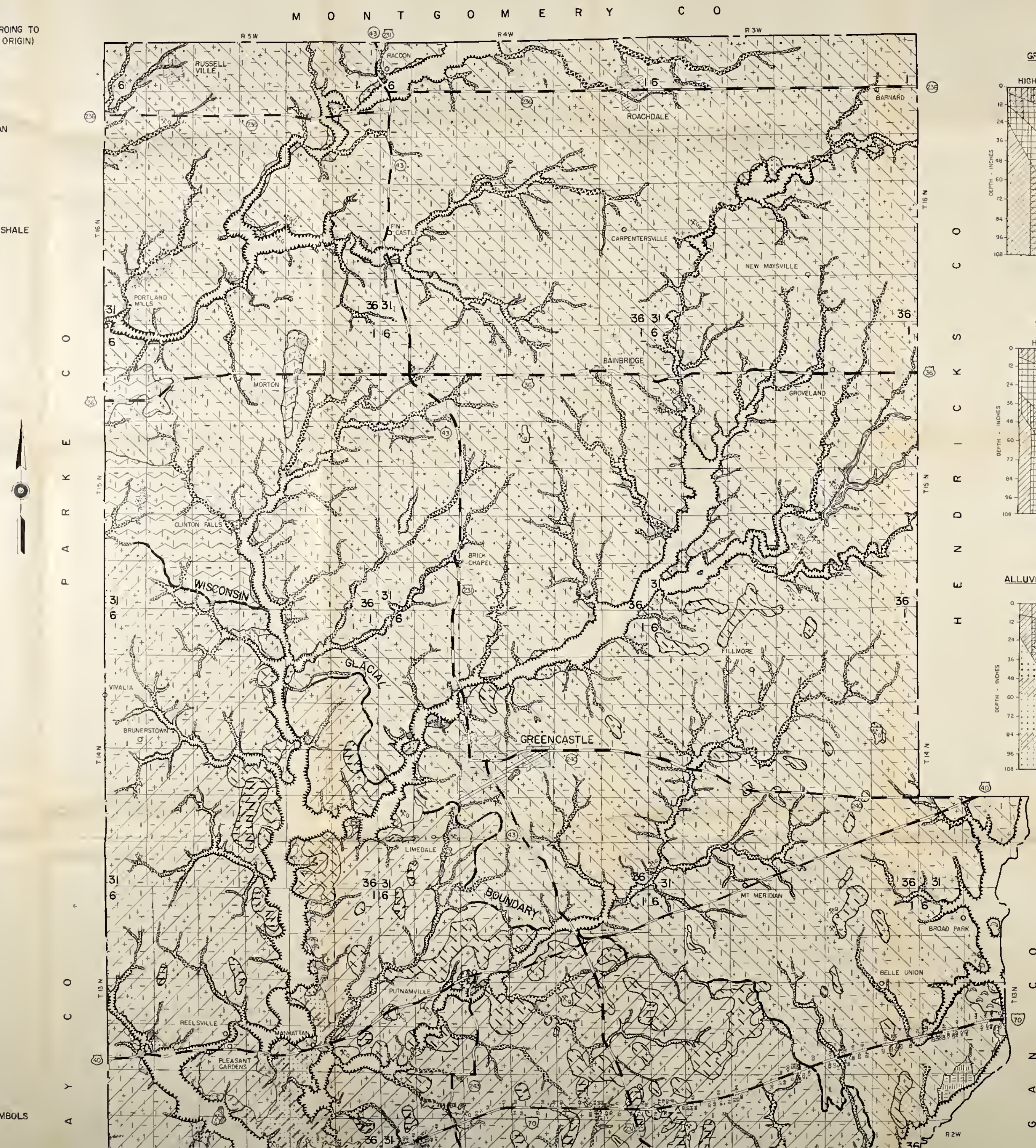
PARENT MATERIALS (GROUPED ACCORDING TO LANDFORM AND ORIGIN)

- GROUND MORAINE, ILLINOIAN
- GROUND MORAINE, WISCONSINAN
- RIDGE MORAINE, WISCONSINAN
- THIN ILLINOIAN DRIFT OVER INTERBEDDED SANDSTONE AND SHALE
- THIN ILLINOIAN DRIFT OVER LIMESTONE
- THIN WISCONSINAN DRIFT OVER LIMESTONE
- ESKER
- SAND DUNE
- ALLUVIAL PLAIN
- OUTWASH PLAIN
- TERRACE
- SLACK WATER TERRACE
- LACUSTRINE PLAIN
- THIN LOESS

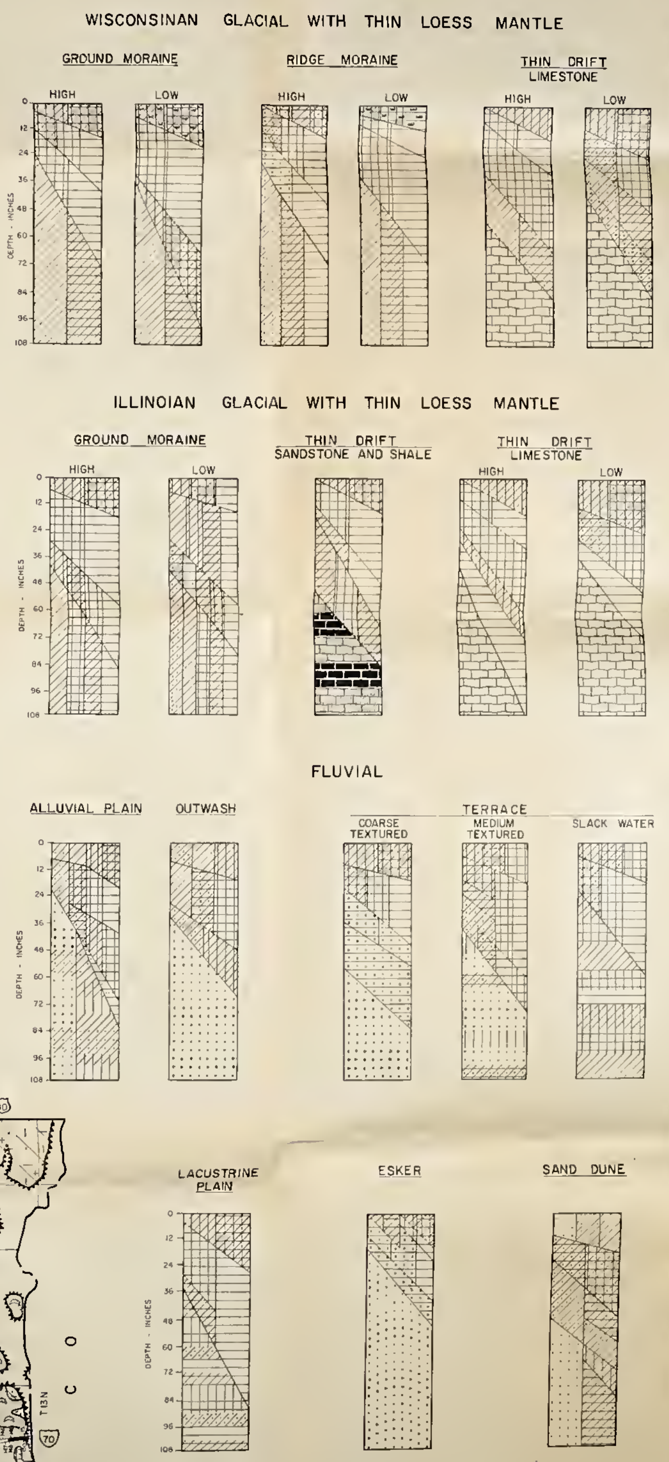
MISCELLANEOUS

- SHALE-SANDSTONE QUARRY
- LIMESTONE QUARRY
- GRAVEL PIT
- LAKE AND POND
- BORING SITES

TEXTURAL SYMBOLS
(SUPERIMPOSED ON PARENT MATERIAL SYMBOLS TO SHOW RELATIVE COMPOSITION)



GENERAL SOIL PROFILES



OUTWASH PLAIN

TERRACE

SLACK WATER TERRACE

LACUSTRINE PLAIN

THIN LOESS

MISCELLANEOUS

SHALE - SANDSTONE QUARRY

LIMESTONE QUARRY

GRAVEL PIT

LAKE AND POND

BORING SITES

TEXTURAL SYMBOLS
(SUPERIMPOSED ON PARENT MATERIAL SYMBOLS TO SHOW RELATIVE COMPOSITION)

GRAVEL

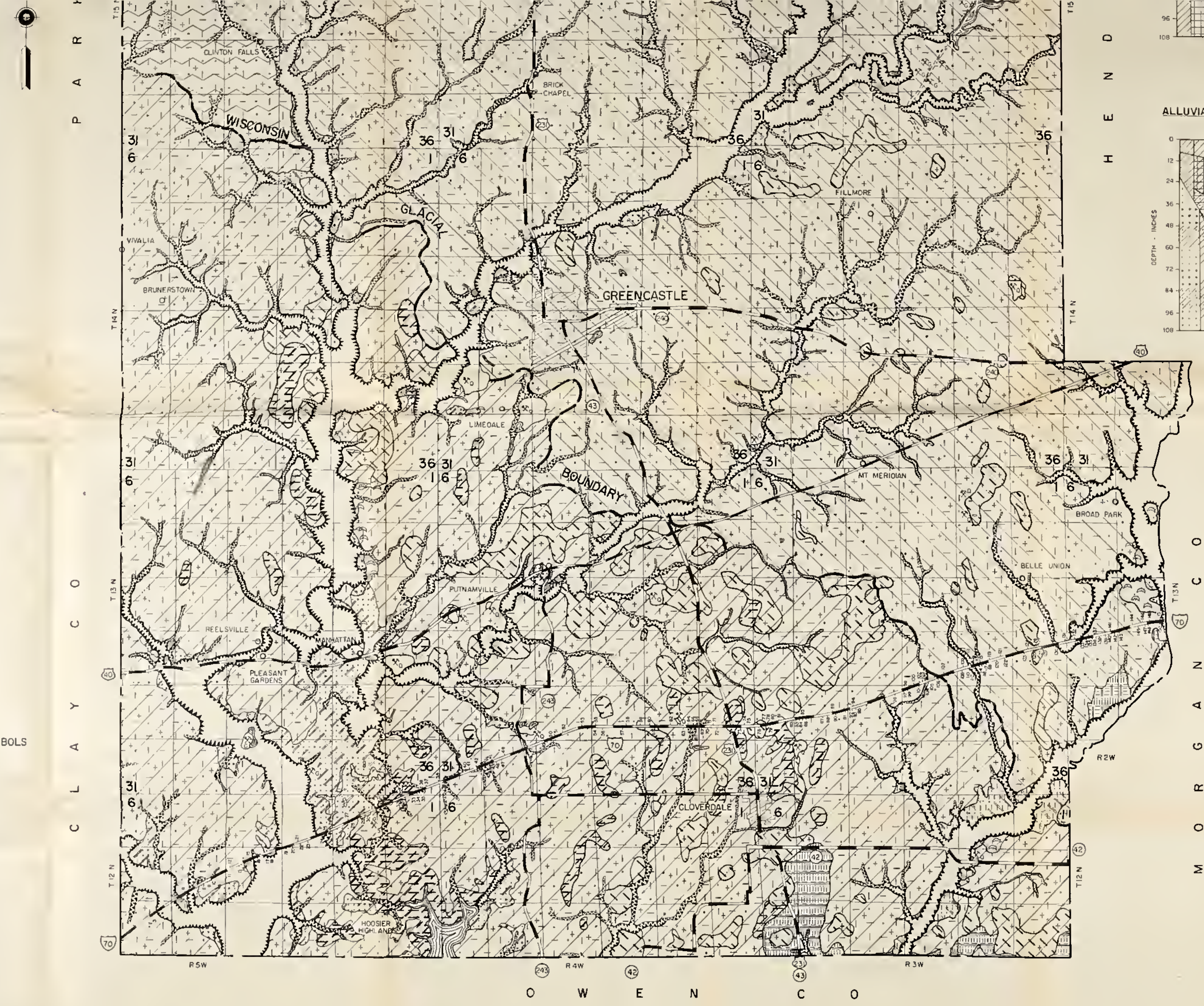
SAND

SILT

CLAY

TEXTURAL SYMBOLS FOR SOIL PROFILES

SILT	CLAY	LOAM
SANDSTONE	LIMESTONE	SAND
GRAVEL	ORGANIC MATERIAL	SHALE



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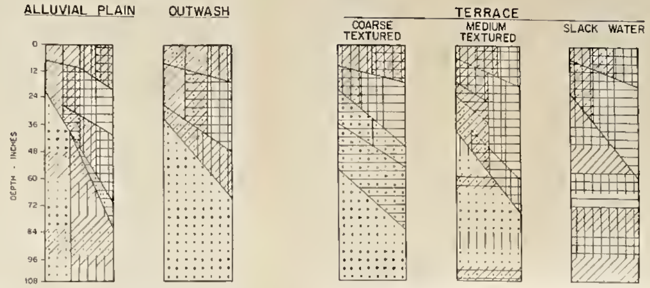

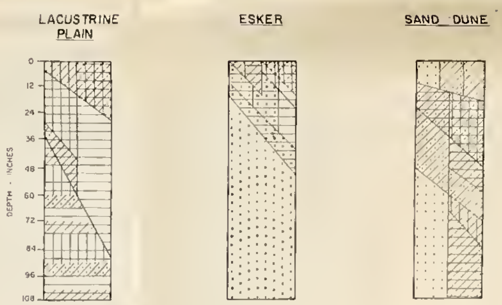

ENGINEERING SOILS MAP
PUTNAM COUNTY
INDIANA

PREPARED FROM
1939 AAA AERIAL PHOTOGRAPHS
BY
JOINT HIGHWAY RESEARCH PROJECT
AT
PURDUE UNIVERSITY
1976

PREPARED UNDER THE SUPERVISION OF PT YEH

SCALE OF MILES
1 3/4 1/2 1/4 0 1 2 3 4

POLYCONIC PROJECTION

DRAWN BY J K BUECHLER, L. FEHO, J. SUTER